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Reinforcing an Old Cantilever Bridge, Philadelphia, Penn.

By Henry H. Quimby*

A very interesting old iron highway bridge of a rare type, carrying 40th St. in the City of Philadelphia over the main line of the Pennsylvania R.R., is being reinforced to extend its usefulness.

It was designed in the office of Joseph M. Wilson, then Engineer of Bridges and Buildings for the Pennsylvania R.R., and was erected in 1874. It is 60 ft. wide and 328 ft. long in three spans—189-ft. main span and two 69.5-ft. anchor spans. There are two trusses 40 ft. apart on centers, and the 10-ft. sidewalks are carried on projecting ends of the floor-beams. The pier support is on riveted columns of plates and angles latticed, one column to each truss on each pier. The columns were encased completely

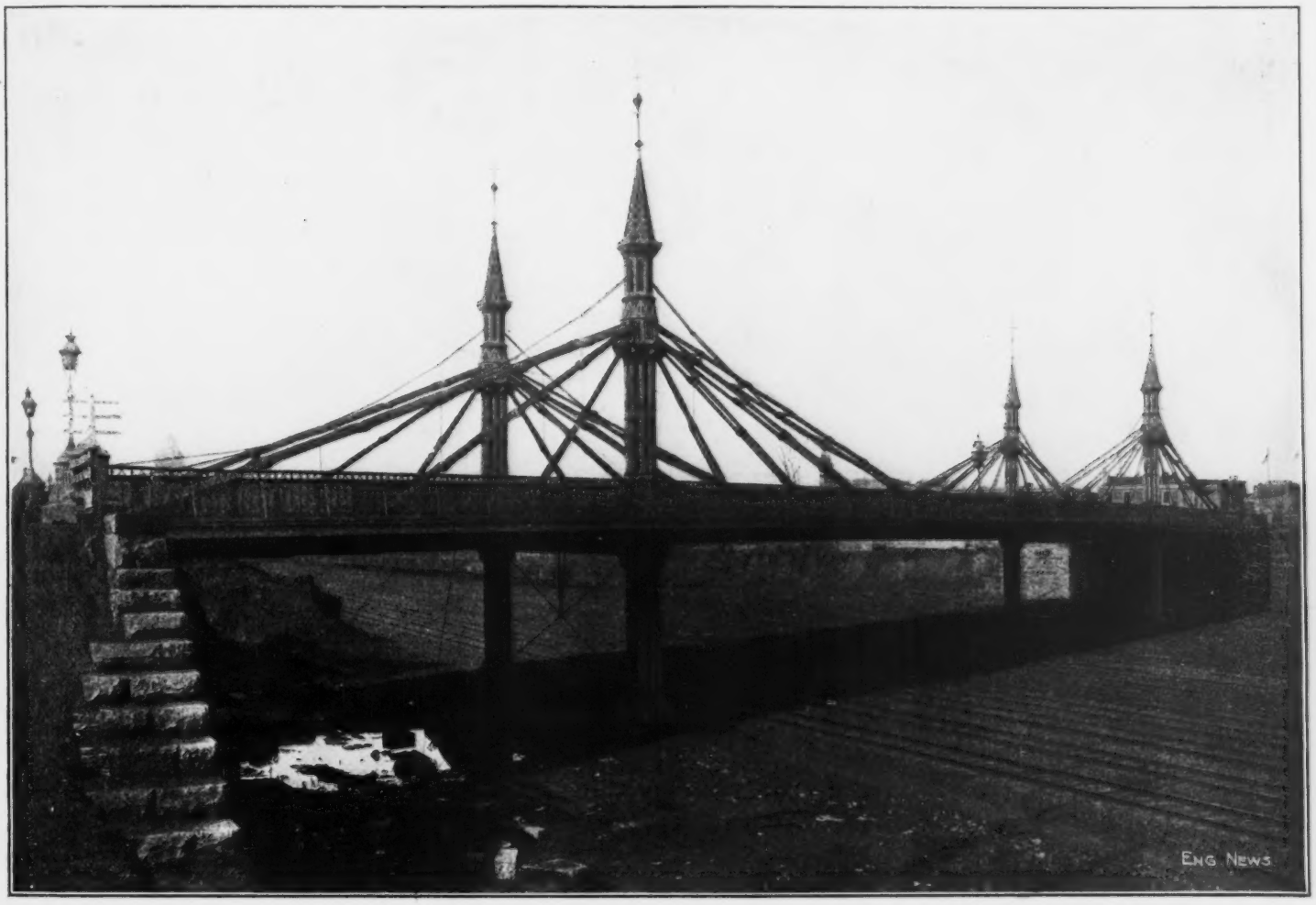


Fig. 1. A Modified Ordish Truss Bridge, Philadelphia, Penn., Now Being Strengthened

The structure is a through-truss cantilever bridge without a suspended span, built on a 30° skew to the railroad tracks. The type is a modification of the "straight link" suspension bridge patented in England, in the year 1857, by R. M. Ordish, the most notable example of which is the Francis Joseph bridge in Prague, Austria, with a main span of about 480 ft., built in 1868. The modification changes the character of the type from a true suspension to a true cantilever, but the 40th St. bridge at the time it was built was called a suspension bridge.

*Engineer of Bridges, Department of Public Works, Philadelphia, Penn.

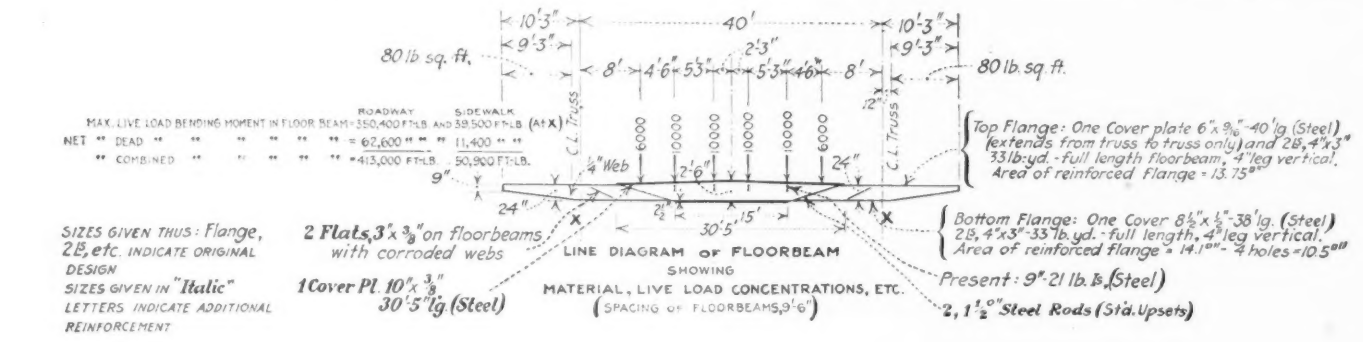
in elaborately ornamental cast-iron shells which were entirely removed long ago, but the accompanying view, from a photograph taken in 1876, shows how the bridge originally looked.

The suspension system consists of eye-bar chains to each panel point from the tops of the main columns, the pin joints of the chains being supported, for the purpose of maintaining straight lines, by a supplementary cable of small eye-bars forming a catenary, carried on a small extension of the main columns. The compression chord is at the floor line and is composed of from two to four lines of 12-in. channels with a 20-in. cover plate,

constituting also the bottom flange of the longitudinal stiffening girder. This girder with the box chord is 54 in. deep—the panels of the suspension system being 21 ft. long—and is continuous throughout the whole length of 328 ft. All temperature adjustment must, therefore, take place at the ends. Provision for this was accordingly made with roller nests at each end on the abutments, and rollers were also placed at the points of bearing of the longitudinal girders in the columns. Rollers were also provided on the tops of the main columns under

slightly, and the laterals have bent ends and in a number of cases were found to be not tight—all elements contributing to possible see-sawing in temperature changes.

At the two points mentioned the effects of temperature movement are seen in the dislodgement of the coping stone through which the anchor bars pass and upon which the end shoes rest. These stones are each pulled forward out of their original position over 2 in., probably because of the bars being choked in the slots. The rollers are under the chords, but the dead-load reaction at each truss


10'3"
40'
10'3"
9'3"
2'3"
9'3"
80 lb sq. ft.
8'
4'6"
5'3"
5'3"
4'6"
8'
80 lb. sq. ft.
MAX. LIVE LOAD BENDING MOMENT IN FLOOR BEAM = 350,400 FT-LB. AND 38,500 FT-LB. (At X)
ROADWAY
SIDEWALK
NET " DEAD " " " " " " = 62,600 " " " 11,400 " "
" COMBINED " " " " " " = 413,000 FT-LB. 50,900 FT-LB.
C.L. Truss
6000
10000
10000
10000
10000
6000
C.L. Truss
Top Flange: One Cover plate 6"x 9/16"-40'lg. (Steel) (extends from truss to truss only) and 2L's 4"x3" 33 lb/yd. - full length floorbeam, 4" leg vertical. Area of reinforced Flange = 13.75"
9"
1/4" Web
2'6"
24"
24"
2 1/2"
30'5"
15'
Bottom Flange: One Cover 8 1/2"x 1/2"-38'lg. (Steel) 2L's 4"x3"-33 lb/yd. - full length, 4" leg vertical. Area of reinforced flange = 14.1"- 4 holes = 10.5"
Present: 9"-21 lb. I-B. (Steel)
2, 1 1/2" Steel Rods (Std. Upsets)
SIZES GIVEN THUS: Flange, 2L's, etc. INDICATE ORIGINAL DESIGN
SIZES GIVEN IN "Italic" LETTERS INDICATE ADDITIONAL REINFORCEMENT
2 Flats, 3'x 3/8" on floorbeams with corroded webs
1 Cover Pl. 10"x 3/8" 30'5" lg. (Steel)
LINE DIAGRAM OF FLOORBEAM SHOWING MATERIAL, LIVE LOAD CONCENTRATIONS, ETC.
(SPACING OF FLOORBEAMS, 9'6")

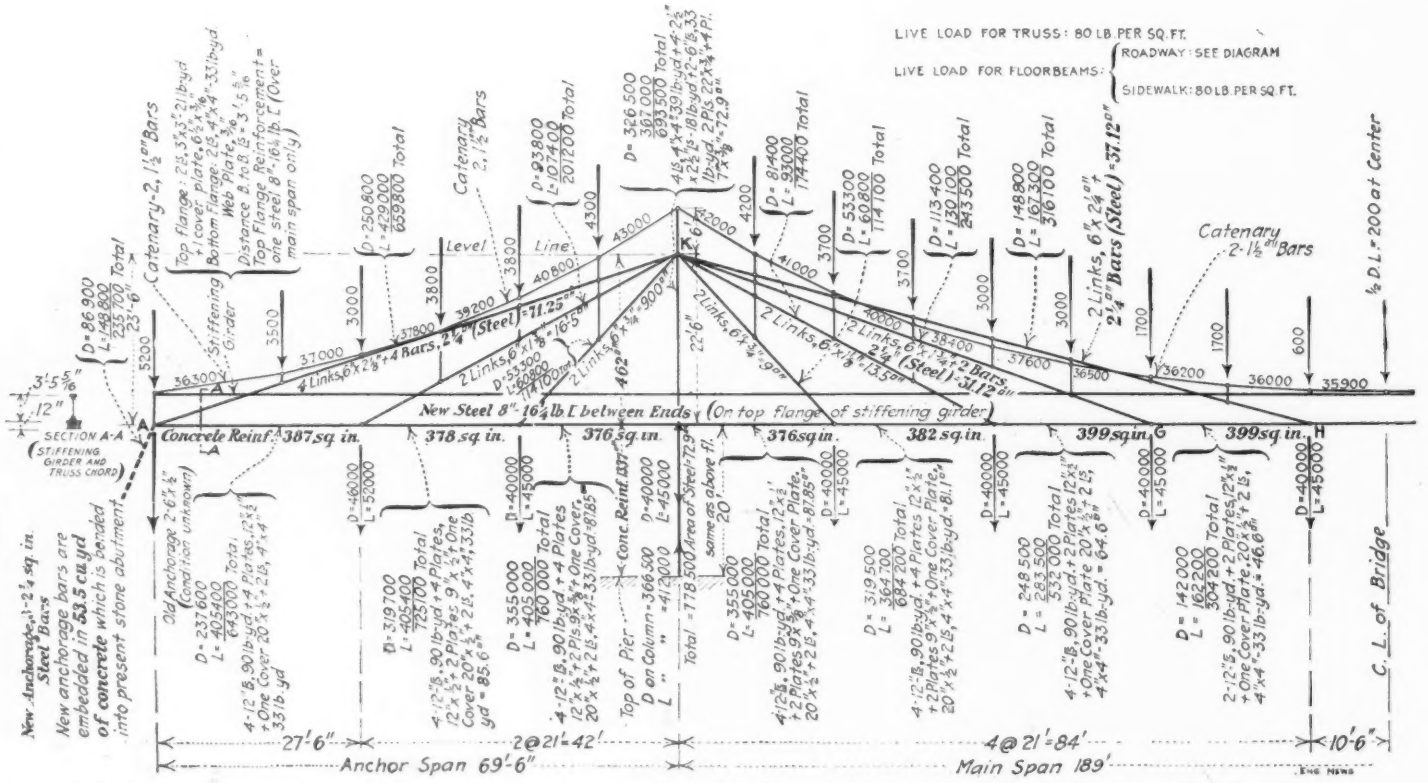

LIVE LOAD FOR TRUSS: 80 LB. PER SQ. FT.
LIVE LOAD FOR FLOORBEAMS: ROADWAY: SEE DIAGRAM
SIDEWALK: 80 LB. PER SQ. FT.
Catenary-2, 1 1/2" Bars
Catenary 2-1 1/2" Bars
Level Line
Stiffening Girder
4 Links, 6"x 2 1/8"x 4 Bars, 2 1/4" (Steel) = 71.25"
2 Links, 6"x 1 1/8"x 16'-5"
2 Links, 6"x 1 1/8"x 13'-5"
2 1/4" (Steel) - 31 1/2"
2 Links, 6"x 2 1/4" 2 Bars, 2 1/4" (Steel) = 37.12"
New Steel 8"-16 1/4 lb I between Ends (On top flange of stiffening girder)
Concrete Reinf. 387 sq. in.
378 sq. in.
376 sq. in.
376 sq. in.
382 sq. in.
399 sq. in.
399 sq. in.
SECTION A-A (STIFFENING GIRDER AND TRUSS CHORD)
Old Anchorage 2'6" x 6" (Condition Unknown)
New Anchorage, 1-2 1/4 sq. in. Steel Bars
New anchorage bars are embedded in 53.5 cu. yd of concrete which is bonded into present stone abutment
Top of Pier
Conc. Reinf. 376 sq. in.
Same as above fl.
Total = 718.500 Area of Steel = 72.9
C. L. of Bridge
½ D.L. = 200 at Center
27'6"
2@21'=42'
4@21'=84'
10'6"
Anchor Span 69'6"
Main Span 189'
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FIG. 2. MAKE-UP AND REINFORCEMENT OF 40TH ST. BRIDGE ACROSS PENNSYLVANIA R.R. TRACKS, PHILADELPHIA, PENN.

the main-cable saddles, and on the tops of the small column extensions under the catenary-cable saddles. There are thus 16 nests of rollers, and the only points intended to be fixed were the bases of the four main columns, which rest on individual stone piers at the level of the railroad below.

Recent inspection made it appear that the only temperature movement of all these expansion points for some time has been at the two diagonally opposite short-corner ends of the trusses, the bridge, as stated, being on a skew. This condition would seem to involve a yielding of the lateral system, and that is not impossible, because the laterals are attached to the light webs of the floorbeams at mid-height and the beams might also swing

end is negative and, therefore, the chords do not bear on the rollers except when the live load is sufficient to overcome the dead uplift. The design contemplated the fixing of the anchor bars in the stone and the sliding of the end pins in horizontally slotted holes in the heads of the bars, the slots being rectangular and the pins passing through the bars in square bushings. There is now no sign of recent sliding anywhere, all the points being apparently cemented together with rust. All the roller nests seem to be immovable, although those on the tops of the columns are in excellent condition as regards corrosion. Those in the columns under the longitudinal girders are rusted fast. As the chord must change its length with the temperature of the air, and the piers

are fixed in their spacing, the columns must be bending back and forth, and they must have been doing so for many years, but evidently no injury has resulted.

As there is no other provision for counter-stresses in the trusses than the longitudinal girders, and the panels are 21 ft. long, the trusses are quite flexible and there is a good deal of vertical vibration under moving loads, every wave of which slightly sways the tops of all the columns. The rollers on the tops of the columns under the main-cable saddles and those under the catenary-cable saddles do not roll on their plates although they are clean, the rolling friction being greater than the resistance of the columns to bending. This springing has been continuing probably from the beginning of the service of the bridge—nearly forty years—and no ill effects can be seen.

The floor-beams were originally suspended from the bottom flanges of the chord channels with four bent straps, 3½x¾ in., riveted so that each beam hung at each end from four ⅞-in. rivets in tension. This detail carried the street traffic—including eight-wheeled street cars that were not thought of when the bridge was designed—for a number of years without any failure. A few years ago the point was reinforced with hangers from the tops of the channels.

The box chords—open underneath and exposed to locomotive exhaust—have become much corroded, as have many of the floor-beams and portions of the eye-bars that enter the chords. The anchor bars cannot be examined without tearing out the ashlar masonry of the abutments, and as it is pretty certain that at least some of them are corroded and reduced in section and in need of reinforcement it was considered economical to supply additional anchorage without any disturbance of the old, thus avoiding the trestling of the span and the reconstruction of the abutments. A number of years ago one of the anchor bars did break, permitting considerable sag of the truss, and was replaced at heavy expense.

The greatest loss of section in the bottom flanges of the floor-beams at any one point is about 2 sq.in. The greatest depth to which corrosion has removed metal at any place on the structure is about ¼ in., which is on the inside of the bottom chord and on some of the eye-bar heads within the chord. It has been assumed that the loss of section in all eye-bars has been 1.75 sq.in. except in eye-bars *AK*, *HK* and *GK* (see Fig. 2), where a loss of 3.5 sq.in. has been assumed, and these latter three were the only bars strengthened in the renewal work noted below.

The reinforcement of the bridge as now being done consists of concrete anchor weights behind the stone abutments, adjustable steel anchor bars embedded in them, adjustable steel square bars strung over the end eye-bar cables to increase their section, concrete filling and enveloping of the box compression chords, steel-channel top-flange cover throughout the stiffening girders, and trussing of the floor-beams. The general details of the old design and of the reinforcement are shown in Fig. 2.

The anchor weights are each 50 cu.yd. in volume, toothed into the backs of the abutments, and thus the new anchorage will be independent of the old anchor bars. The provision for expansion is in the embedment of the new anchor bars in a viscid body of coal tar, which will flow around the bars as they are slowly moved with the end of the chord. The bars were first surrounded with a small body of cement concrete to protect them, then about 1 in. of plastic tar over the concrete, then the concrete weight was cast. The radius of movement of the tops of the bars is about 10 ft., which is expected to allow a sufficiently gentle curvature to prevent injury to the bars from the bending. Because the anchor bars have some inclination from the vertical the expansion and contraction of the truss chords will slightly raise and lower the skewback of the new rods and consequently change somewhat the distribution of stresses between the new and the old truss members, and thus affect the dead-load deflection of the main span. The total length of the new reinforcement is sufficient to make the effect of the variable stress negligible.

The concrete filling and enveloping of the compression chord will be done after the tightening of the new truss rods and before the wood floor is placed, the objects being to give to the new bars their full share of stress, and to permit the concrete to get as much as possible of the dead-load stress in the chord. Before the wood floor is placed the ironwork will have about one-half of the dead-load as initial stress, and after the concrete has hardened the balance of the dead-load and all of the live-load will be carried by the two materials jointly, the share of each being in proportion to their moduli of elasticity and their relative areas of section. The effect of stress will be to shorten the chord, and neither material can shorten without the other, so each will necessarily carry according to its strength.

The reinforcement of the top flange of the longitudinal girder will be a single 8-in. steel channel spliced to be continuous throughout the whole length, and riveted to the girder through holes drilled in the field by electric power. Because of the continuity of the member this channel will be subject to both tension and compression at different points.

The scheme of reinforcement includes also the concreting of the main columns. Above the floor they will be merely filled, but below the floor they will be filled and incased. It seems probable that the apparently inevitable bending at the floor line will crack this concrete somewhere but it should not be harmful, the bending being doubtless in a more or less gentle curve, and the concrete will certainly increase the carrying capacity of the columns.

The design of the truss is one that is inherently limber, as is a suspension bridge, and the reinforcement will not materially correct this defect because it does not change the system of trussing—it merely makes the stiffening girder a little stiffer and adds section to the truss members. The work is purely a repair job, replacing strength lost through corrosion, and adding some to provide for carrying an additional line of street railway. The bridge is a wooden-floor structure and, therefore, not regarded as a permanent one, but with proper care and periodical renewal of the floor it will last indefinitely. The repairs will cost—including some new floor—about $16,000. A new bridge with concrete floor and permanent paving would cost about $80,000. The cost of maintaining the reinforced old bridge—painting the iron and renewing the floor—will be less than the interest on the excess cost of the new bridge. Besides this, the repairs could be made from an existing fund, and the money for a new bridge was not so easily obtainable.

The Development of Balancing Devices for Centrifugal Pumps

By Alex. V. Mueller*

Within the last five years centrifugal pumps have found their way into fields formerly served exclusively by piston pumps. Many difficulties had to be overcome, especially with high-pressure pumps, and, aside from hydraulic efficiency, reliability in operation had to be attained. To this end, the most important and, unfortunately the most difficult, problem to solve was to care for the axial thrust of shaft.

Obviously, two general methods of caring for this thrust are open for use: (1) receiving the pressure on adequate bearing surfaces, and (2) balancing the force by counter pressures in balancing devices. Thrust bearings are undesirable because inefficient. The most important balancing devices originated in Europe where they

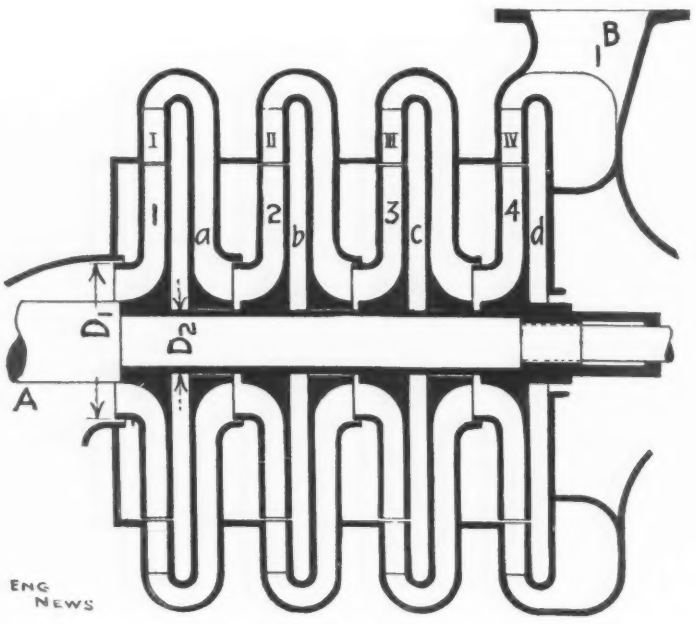

Fig. 1

are used extensively. Up to the present they have been less used in this country. While there are many American patents on balancing schemes, none have gained any great prominence.

End thrust is due to several actions: (1) the unbalanced pressure on the impeller opposite the inlet opening; (2) the change in direction of water entering the impeller axially and leaving radially necessitates an axial force transmitted along the shaft to the impeller, and (3) the different pressure conditions on the inside of the impeller and in the idle spaces outside it. The forces due to (1) and (3) tend to counterbalance.

Fig. 1 shows a cross-section of a multi-stage pump where impellers are marked *1, 2, 3* and *4*; guide passages *I, II, III* and *IV*; suction inlet *A*; discharge outlet *B*. The impellers are of the side-suction type.

The area upon which the unbalanced pressure acts is expressed by the equation

$$S = [D_1^2 - D_2^2]\frac{\pi}{4}$$

If p represents the pressure created by one impeller and kp is the unbalanced part of this pressure acting upon S, it is easy to derive the formula

$$P = n\,k\,p\,S, \qquad (1)$$

in which

P = the axial thrust;

n = number of stages.

This formula deals with the static forces only, and does not take into consideration the dynamic ones, such as losses through clearances, creation of impeller paraboloid and dynamic action of shocks due to change of flow from an axial to a radial direction.

It cannot be emphasized too strongly that these dynamic forces preclude exact calculations. Even if theoretical calculations could be correctly made, inaccuracy in workmanship, etc., would offset them. Most designers have probably observed that pumps, even when made from the same pattern, show marked deviations from each other on test. Then too, after a short period, unequal wear, for instance, would make different leakages, which would immediately cause end thrust.

It may be well to mention at this point that double suction impellers are quite frequently considered to be in hydraulic balance. Owing to their symmetrical construction this supposition is only theoretically correct, as the foregoing paragraphs show. There are, in fact, double-suction pumps in operation that give enormous end thrusts.

It was soon realized that it was a fruitless task to take care of the entire thrust load by stationary thrust-bearings, especially in high-pressure turbine installations, where the end thrust assumes uncontrollable proportions; a multitude of balancing devices were originated, many of them ending their career at the testing plant.

The various devices can be classified, according to the results obtained, into (1) partial- and (2) complete-balance types.

Partial-Balance Devices

This class is able to neutralize only part of the existing end thrust. No matter how small the remaining unbalanced thrust may be, a positive thrust bearing must be added to take care of it.

Fig. 2 shows the well known old Sulzer arrangement for centrifugal pumps. This was used by them with marked success for many years, and until they substituted the restless balance. The impellers are arranged in pairs, taking suction at opposite sides, and the axial thrust would, theoretically, be entirely eliminated if there was no leakage between the different stages, which creates an axial thrust toward the left. Since impellers are arranged in pairs, they are, of course, just as liable to develop some unforeseen end thrust as the double suction pump.

The most logical and the simplest way to obtain partial balance was to make use of the areas shown in Fig. 1.

$$[D_1^2 - D_2^2]\frac{\pi}{4}$$

Fig. 3 shows an impeller with balance rings on the suction as well as on the opposite side having equal sealing fissures *a* and *b*. On the rear of the impeller, two chambers are formed, *1* and *2*, and by connecting chamber *2* to the suction inlet, partial balance is obtained. The packing rings may be varied in diameter, and any

*Mechanical Engineer and Superintendent, Kingsford Foundry & Machine Works, Oswego, N. Y.

predetermined axial force can be taken care of. The vulnerable point in this construction is that the balance is completely offset by wear and tear, when the stationary thrust-bearing will have to take care of excess thrust and is very liable to be destroyed.

Fig. 4 shows the same principle applied to multi-stage pumps, the connection between front and back of impeller being made by holes *a* through the impeller web.

The same effect is obtained in the arrangement shown in Fig. 5, where the chambers *a* and *b* are connected by slides in hub, either in straight or spiral direction. This arrangement avoids, to a certain extent, the eddies which are created through openings like *a* in Fig. 4.

In Fig. 6 the impellers have packing rings on both sides, as before. The stationary walls *w*, however, do not extend clear down to the shaft; consequently the same pressure will prevail on the back side of the first impeller as on the suction side of the second impeller, and by connecting the chamber *r*, composing the last stage, with the suction chamber of the pump, partial balance is obtained.

Fig. 7 shows the well known Rateau arrangement which, however, has been long abandoned. The impeller disks are of different diameters; the one opposite the suction inlet being the smaller, with the object of obtaining less pressure in the chamber r_1 than in chamber r_2. This amount represents the circular area from *a* to *b*. If the diameter of disk in chamber r_1 is figured carefully, the end thrust is practically eliminated.

Fig. 8 embodies the same idea, but this arrangement must necessarily result in still lower pump efficiency, since the extension disk on the suction side will create heavy friction losses.

All of the above mentioned arrangements deal with individual balancing of each impeller. The balancing, however, can be accomplished by a single membrane, located either inside or outside of the pump. Before taking up this type of balance, the value of end thrust must be discussed briefly further.

Equation 1 shows the influences which the coefficient k has upon the value of axial thrust. The coefficient k changes with the conditions under which the pump is working. For illustration, suppose that at normal operation where the pump works at its highest efficiency, the coefficient k is equal to $\frac{2}{3}$. However, when the discharge valve is closed, or nearly closed, this value of k will increase approximately to 1, since in this case the total pressure must be created by the impeller itself, as the diffusor does not add pressure except when a certain amount of water is passing through the same. Very little water is passing when the discharge valve is closed; only the leakage through packing fissures. (Further consideration of changes to which the coefficient is subjected is omitted as it would be of no special interest in this article).

Let *a* be the atmospheric pressure in pounds, *h* the manometric suction lift, expressed also in pounds, then the pressure in chambers *a*, *b*, *c* and *d*, Fig. 1, would be;

$$kp + a - h,$$
$$[k + 1]\ p + a - h,$$
$$[k + 2]\ p + a - h,$$
$$[k + 3]\ p + a - h.$$

The pressure in chamber *d* for *n* stages would be

$$[k + n - 1]\ p + a - h$$

for normal operating conditions.

If this pressure is used to work upon a balancing disk located inside or outside of the pump, then the size of the balancing disk should be;

$$nkpS = [(k + n - 1)\ p + a - h - a]\ S_1$$

$$S_1 = \left[\frac{nkp}{(k + n - 1)\ p - h}\right] S \qquad (2)$$

This formula refers to a case where the space back of the balancing disk is subject to atmospheric pressure. However, if this chamber is connected to the suction line, or if the water flows to the pump under atmospheric pressure, the membrane *h* is eliminated, and Equation 2 changes into;

$$S_1 = \left[\frac{n \times k}{k + n - 1}\right] S \qquad (2a)$$

This formula shows that the value of S remains constant under changed conditions in a single-stage pump when n is equal to 1, while it changes for multi-stage pumps with the value of k.

This k may be chosen, for example, to equal $\frac{2}{3}$ in a four-stage pump for normal operating conditions, and with Equation (2a) applied there will be:

$$S_1 = \frac{4 \times \frac{2}{3}}{\frac{2}{3} + 4 - 1} S = 0.727\ S$$

while if k, for instance, should equal $^4/_5$, S_1 is equal to $0.85\ S$. If, therefore, the area of balance member would be designed for normal operating conditions where k is equal to $\frac{2}{3}$, this area would not be sufficient if k becomes equal to $^4/_5$, which is the case in starting the pump. S would be too small;

$$\frac{0.85 - 0.727}{0.727} = 16.5\%$$

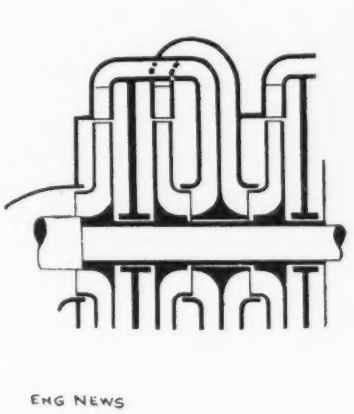

FIG. 2

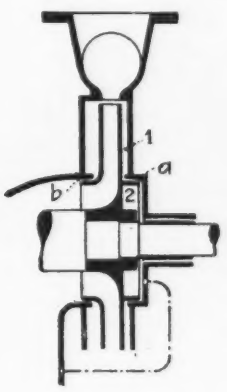

FIG. 3

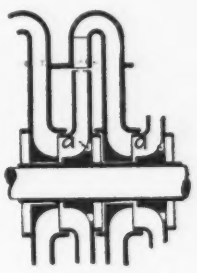

FIG. 4

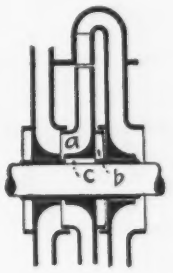

FIG. 5

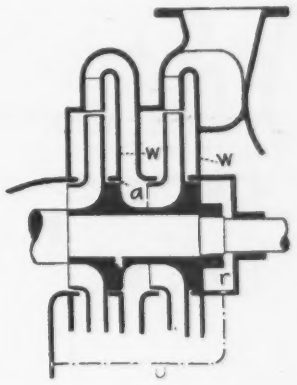

FIG. 6

A heavy axial thrust toward the suction side would result.

While there is, without doubt, a great advantage in balancing with one member instead of balancing each individual stage, there is the disadvantage in that a positive thrust-bearing has to be provided in order to meet changing conditions during operation. The simplest arrangement would be to mount a piston K, as the last member of the pump interior.

Fig. 9 shows the early Schwartzkopff type of balancing, where the effective balancing area is calculated for normal operating conditions, while the remaining unbalanced end thrust is taken care of by a stationary thrust-bearing.

The changing value of axial thrust can be effectively provided for by intentionally selecting a piston too large, and, instead of connecting the chamber at back of piston directly to atmosphere, a valve can be installed, which has to be manipulated by hand, obtaining thereby the exact pressure to counterbalance the end thrust. Since the regulation has to be done by hand, this kind of balancing must be classified under the partly balanced devices.

The same may be said of Fig. 10 where the pressure created by the last stage is conducted to the ring chamber r, located at suction side of first impeller. The correct pressure is obtained by a throttling valve.

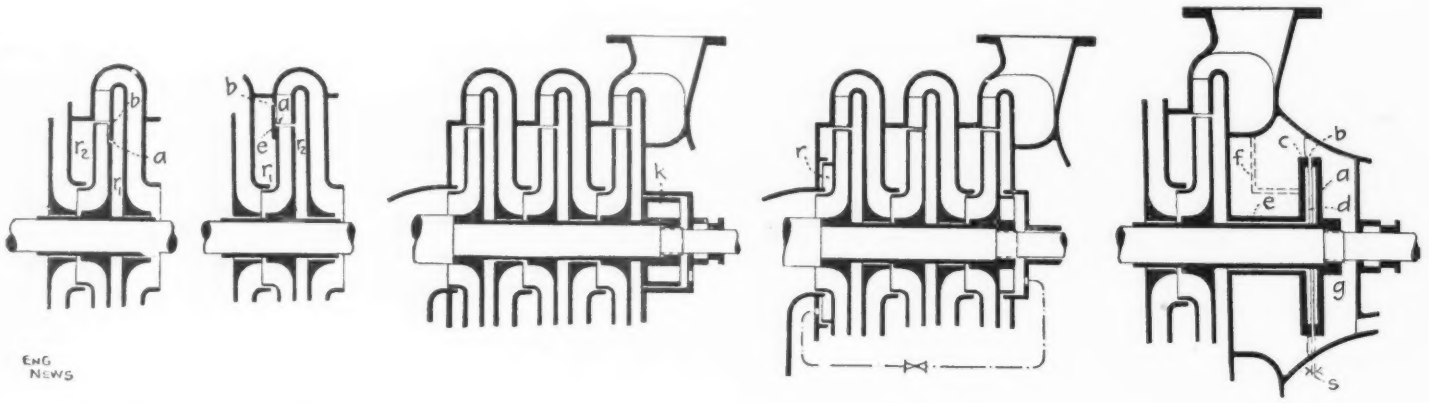

Fig. 7. Fig. 8 Fig. 9 Fig. 10 Fig. 11

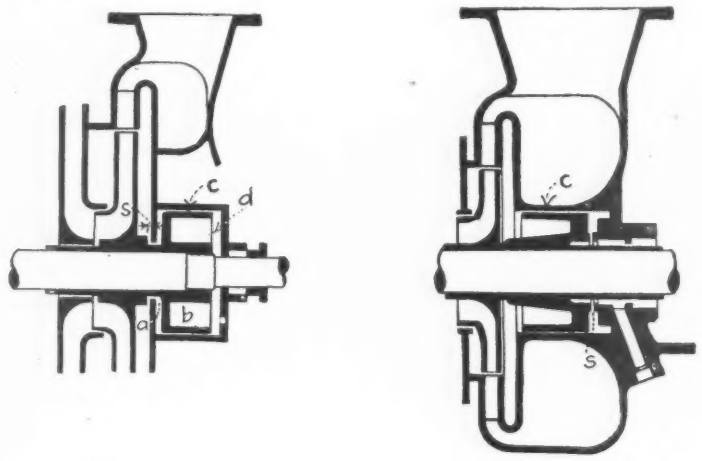

Fig. 12 Fig. 13

Complete-Balance Devices

A complete-balance device has to be able to adjust itself automatically to all changes that might arise during operation. It has to be such that all axial thrust is counterbalanced without any auxiliary thrust-bearing, i.e., the rotating parts have to float, and find a center in a similar manner as the armature of an electric motor finds its magnetic center.

Many important inventions have been made at almost exactly the same time by men living in different portions of the world. This was the case with automatic restless balancing, which was successfully solved theoretically and mechanically by Sulzer Bros., and Schwartzkopff. It is difficult to say which one succeeded first. All former makeshifts could be at once abandoned and a fully reliable pump put out.

The ideas of Sulzer Brothers and Schwartzkopff were identically the same:

The connection of a practically constant throttling fissure with another throttling fissure very sensitive to the least axial motion of the shaft, having between these throttling fissures a regulation chamber, the pressure in which acts upon the balancing device.

The designs on this principle have been perfected so that pumps can work against a 3000-ft. head without any auxiliary stationary thrust-bearing. They are used in mine work, where reliability is the most essential factor.

Fig. 11 shows the Sulzer arrangement. A disk a is mounted as the last member of the interior of the pump, and forms, with the stationary wall c, a chamber d which is connected by cylindrical fissure e with pump pressure, while the radial variable fissure s leads to the atmosphere. The chamber d may be connected to pump pressure through f instead of through the throttling fissure e. The chamber g, into which the fissure s leads, is connected to the atmosphere.

The fissure s changes with axial motion of shaft, thereby increasing or decreasing the resistance of the

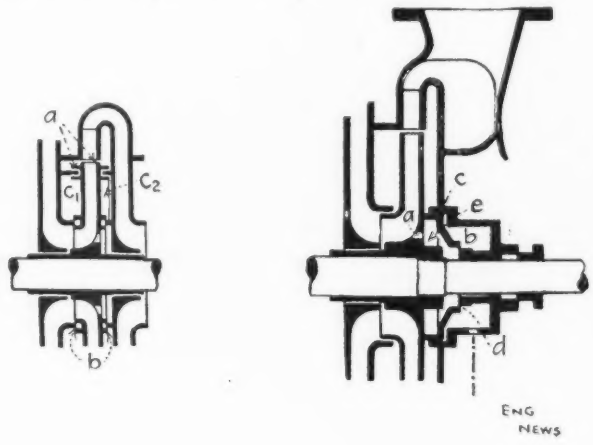

Fig. 14 Fig. 15

fluid passing through this fissure, while the horizontal throttling fissure *e* remains practically constant, on account of its considerable length.

If the shaft moves toward the left, the fissure *s* reduces; consequently the resistance offered to the passing water is increased and therefore the pressure in chamber *d* increases. Again, if the shaft moves toward the right, the fissure *s* opens, the resistance offered to the water in this passage diminishes, and the pressure in *d* decreases. The chamber *d* will always have the necessary pressure to keep the running parts in equilibrium.

Fig. 12 shows the balancing of Schwartzkopff, which is, as mentioned above, similar to the balancing in Fig. 11, except that the vertical throttling fissure is located as close as posible to the center of shaft, which construction has many mechanical advantages.

The maximum pressure in the pump passes through the throttling passages *s*, and acts in the chamber *a* upon a balancing piston *b*. The clearance *c*, between this piston and the stationary piston sleeve, forms a constant throttling fissure, and connects the pressure chamber *a* with the chamber *d*, which leads to the atmosphere.

as a balancing medium, and they are provided with double packing fissures *a* and *b*, the former near the circumference of the impeller, and the latter near the suction inlet. The outside rings on the rotor overlap the stationary cylinders for a considerable length when in mid-position, and the axial displacement of rotor to either side influences their throttling effect very little. Different, however, is the effect of the inner throttling rings *b* which are so arranged that they are not in contact with the stationary corresponding grooves when the impeller is in mid-position. Therefore, as soon as the axial displacement occurs, they cause a throttling effect upon the fluid on one side. A motion toward the left creates an increase of pressure in chamber c_1, and a decrease in c_2, while a motion toward the right has the reverse effect. This balancing is complete and restless but sluggish in meeting momentary thrusts.

Fig. 15 illustrates an arrangement where the impeller is partly balanced by the usual holes through the web *a*. In order to take care of the remaining unbalanced thrust, a chamber *b* is located behind the last impeller. There are two throttling fissures, a constant throttling

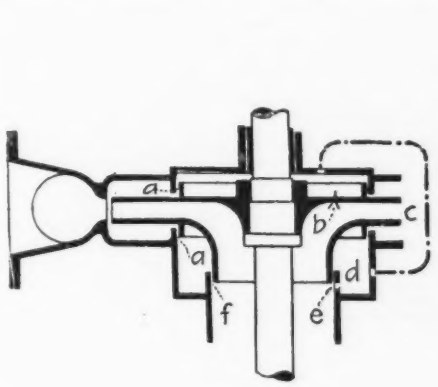

Fig. 16

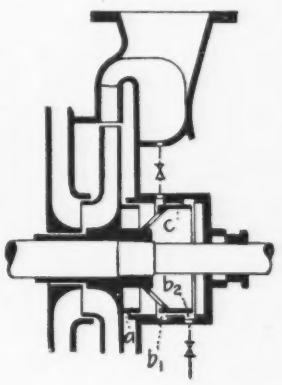

Fig. 17

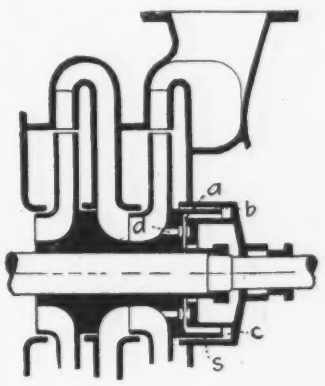

Fig. 18

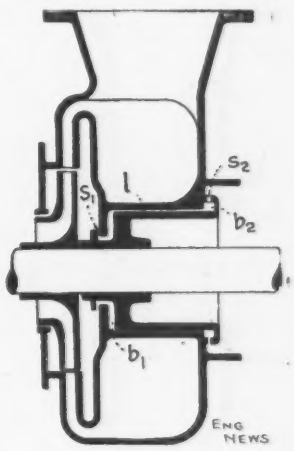

Fig. 19

When the pump starts, the whole shaft moves toward the right, or pressure side, and the regulating disk on the shaft tends to close the throttling passage until the fluid pressure in chamber *b* is counterbalancing the axial thrust.

With the throttling fissure *s*, located near the center of the shaft the two faces of this passage have to stand farther apart than on a circle of larger diameter, and this results in two advantages. It is (1) not too sensitive and (2) has long life.

The water leakage from the balanced piston is very small, and in large units is used for supplying water for water-cooled bearings; in this instance the mechanical efficiency is far beyond what can be obtained with a stationary thrust bearing.

In Fig. 13 the order of throttling fissures is reversed; the constant fissure *c* is located ahead of the variable fissure *s*, which arrangement has some mechanical advantages.

Fig. 14 illustrates one of the various Rateau devices where the impellers are balanced individually. The balancing is entirely automatic, but on account of mechanical difficulties and heavy friction losses, the construction is self-condemned.

The usual relief holes through the impeller hub are again in evidence in Fig. 14. Impeller disks are used

fissure *c* and a variable throttling fissure *d*, all of cylindrical formation. Since the chamber *e* is connected with suction by the above mentioned holes *a*, there will be a constant return flow from *b* to *e*, according to the throttling effect of *d*, which is greater when moving toward the right, smaller when moving toward the left; the pressure in the regulating chamber *b* will always be such as to keep the impeller in balance.

Fig. 16 illustrates a similar American arrangement, by Hunsaker, applied to vertical shafts. The impeller has packing rings of equal diameter on both sides, and the chamber *b* is connected by piping *c* with the suction side of impeller. In order to get the automatic effect, and to take care of the weight of impeller and shaft, a regulating chamber *d* is provided. The variable throttling fissure in this instance is made by the impeller sleeve *f*, and a number of holes *e*. According to more or less overlapping of this sleeve *f* over the holes *e*, the pressure in the chamber *d* will increase or decrease respectively, and keep the impeller in position. The arrangement of holes *e* offers, however, some mechanical difficulties, especially with muddy water.

A skillfully arranged, but somewhat too complicated, American arrangement (by Kretzschmar) is shown in Fig. 17. The practically constant throttling fissure is located at *a*, while there are two variable throttling fis-

sures located at b_1 and b_2. These throttling grooves are connected to different pressures, and are opened or closed by a rotating piston c. If the impeller, on account of large thrust, moves toward the left, the groove b_1, which furnishes high pressure, will be closed while the groove b_2, which furnishes low pressure, will open wider, and the pressure in the regulating chamber d will decrease. This will result in the rotating parts moving toward the right; the opposite effect is obtained if the impeller first moves toward the right. Fig. 17 has the advantage of a double effect over Figs. 15 and 16.

Fig. 18 shows a device similar to Fig. 12 with two throttling fissures, a constant one b, and a variable one s, with the regulating chamber c between. If the shaft moves toward the right, the clearance s is reduced, and pressure in c increases. When the shaft moves toward the left, pressure in c decreases; naturally the impeller will find its equilibrium. The leakage flows through holes d back into suction.

Finally two more of the many interesting Schwartzkopff balancing devices should be mentioned. Fig. 19

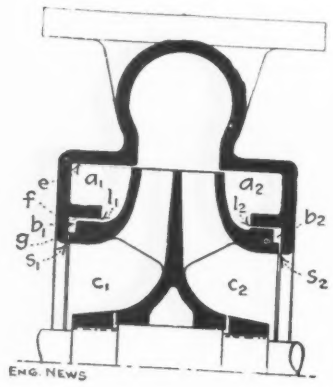

FIG. 20

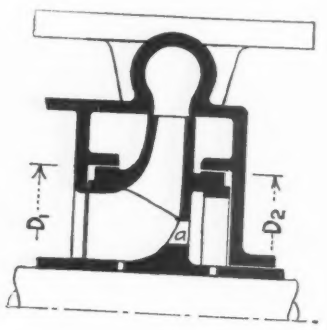

FIG. 21

shows a balancing device for large pumps, and for such as have to handle gritty and muddy liquids.

The arrangement shows two vertical throttling fissures, s_1 and s_2, and one constant throttling passage l in connection with two regulation chambers b_1 and b_2. The action is similar to the one shown in Fig. 12. Should the piston and sleeve wear whereby the leakage through l could increase to such an extent that necessary balancing pressure cannot be obtained in b_1, then the running parts would tend to drift toward the left; but at this moment the second throttling fissure s_2 comes into action, and by the resistance offered to the fluid in passing through the same, not only will pressure in b_1 increase, but also in the regulation chamber b_2, which adds a large balancing area. The throttling fissure s_1 will ordinarily open only a small fraction of an inch, and will therefore act as a filter, keeping gritty substances out of the main balancing apparatus.

This may be called the most perfect and reliable automatic balancing arrangement known today.

Fig. 20 is a practical and intelligent modification of multi-stage pumps, of the Schwartzkopff balancing, shown in Fig. 12 applied to individual impellers. In the case of a double-suction impeller, both sides are symmetrical; each having a constant throttling fissure l_1 and l_2, and a variable one s_1 and s_2.

Assume that the impeller moves towards the left; the throttling fissure s_1 closes, while s_2 opens; therefore, pressure in a_1 increases, and in a_2 diminishes. This tends to move the impeller toward the right side, and ordinarily it will find its equilibrium after a few oscillations. The balancing just described would be complete without taking into consideration the balancing chambers b_1 and b_2, simply by the change of pressure in the large spaces a_1 and a_2.

Studying the chambers b_1 and b_2, it will be found that they have a peculiar mission—when impeller is in mid-position they may be considered as being in easy connection with suction chambers c_1 and c_2. However, this picture changes as soon as impeller leaves mid-position. Let the impeller move toward the left; this closes the fissure s_1, so that very little water will pass back toward the suction. Naturally, the area from e to f will be exposed to maximum pump pressure and, in addition, the area of balancing chamber b_1, from f to g, will also be exposed to maximum pump pressure. On the other side just the reverse will happen; the balancing chamber b_2 will belong decidedly to the suction chamber c_2, since s_2 is open and leakage through throttling l_2 passes without resistance into suction. This arrangement, therefore, will balance under almost any condition, and is not influenced by unequal wear and tear of the throttling fissure; even if one side should be entirely closed by some foreign matter becoming caught in the vanes, for instance.

This makes the arrangement suitable for side-suction pumps, as sketched in Fig. 21. The back of the impeller is connected with the suction by holes a. In order to compensate for the natural tendency of thrust toward suction the two balancing rings D_1 and D_2 are advantageously chosen of different diameters. Beside balancing features, it may be pointed out that there are also many advantages from the manufacturing standpoint, since no additional mechanical devices are required, and up to the present this may be called the most sinple and perfect balancing for single-stage pumps. It goes without saying that this arrangement could also be utilized for turbine pumps.

In conclusion, a comparison may be made between these several different balancing devices which have been described.

The loss of pressure betwen two chambers separated by a fissure of length l and width s is:

$$\Delta p = \frac{V^2}{2g}\left[n + \zeta \times \frac{2 \times l}{s}\right]$$

In which V equals velocity of water through fissure, n and ζ are constants derived from experience, and g is the familiar gravity acceleration factor 32.16.

As the axial motion of centrifugal pumps has to be limited to a minimum—to a small fraction of an inch—it is apparent that devices where l changes are sluggish in action and far inferior to the ones where s changes: the latter act quickly and accurately. This width s changes in Figs. 11, 12, 13, 18, 19, 20 and 21, and, therefore, they are to be preferred to the ones where l changes, Figs. 14, 15, 16 and 17.

The entirely automatic devices have attracted a good deal of attention within the last few years as they have the advantage over incompletely balanced devices in that they rid the pump of such members as marine thrust collars, ball bearings, etc., which are a more or less constant source of trouble.

The Selection of Explosives for Engineering Work

The characteristic features of the principal explosives used in mining and engineering operations are discussed in Bulletin 48 of the Bureau of Mines, Department of Interior, Washington, D. C. The following extracts from this Bulletin are of especial interest to engineers:

It is generaliy recognized that in driving tunnels, sinking shafts, and in removing hard rock in metal mining, explosives having a high disruptive force are preferable. In work of this kind the cost of drilling holes is an important item. It has been found more economical to drill a few holes and to load them with an explosive of high disruptive force than to drill a number of holes and use weaker and cheaper explosives. It is important that explosives used in this class of work should produce the minimum amount of poisonous gases.

The only available explosives which develop the required disruptive force for removing hard rock in metal mining and tunneling work, are "straight" nitroglycerin dynamites and gelatin dynamites. Black blasting powder, granulated nitroglycerin powders, low-freezing dynamites, and ammonia dynamites are not suitable or satisfactory for work of this kind, as they do not develop the requisite disruptive force; furthermore, these explosives, in common with the straight nitroglycerin dynamites, produce poisonous gases.

The gelatin dynamites have been used to a large extent in wet blasting, as in the removal of obstacles to navigation and in deep workings, and as a general rule are best suited

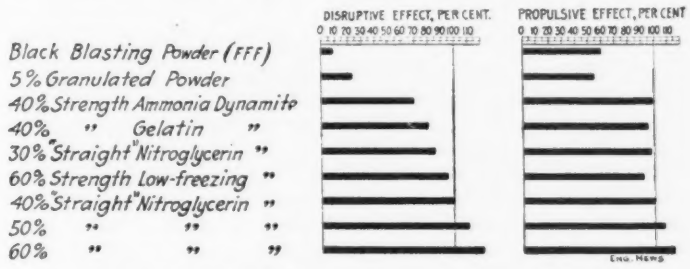
DISRUPTIVE EFFECT, PER CENT.
0 10 20 30 40 50 60 70 80 90 100 110
PROPULSIVE EFFECT, PER CENT
0 10 20 30 40 50 60 70 80 90 100 110
Black Blasting Powder (FFF)
5% Granulated Powder
40% Strength Ammonia Dynamite
40% " Gelatin "
30% "Straight" Nitroglycerin "
60% Strength Low-freezing "
40% "Straight" Nitroglycerin "
50% " " "
60% " " "
Eng. News

DIAGRAM OF THE COMPARATIVE ENERGIES OF COMMERCIAL EXPLOSIVES

for these purposes. In the manufacture of these dynamites the nitroglycerin is gelatinized by the addition of a small percentage of nitrocellulos. The jelly-like mass thus formed is impervious to water. By the addition of different percentages of suitable absorbents, the various grades of these dynamites are made.

The selection of an explosive for submarine work is of special importance because of the severe moisture and temperature conditions under which it must be used, also because of the great expense incurred in preparing every blast, a consideration that makes any failure of importance. It has also been generally recognized that when used for blasting under water, explosives produce less useful work than when used for excavating on land. This decrease is due to the reduction of the temperature of the products of combustion and to the confining effect of the water on the material to be blasted, an effect that more than counteracts the beneficial effects of the water stemming.

Four different classes of explosives were used in comparative-efficiency tests. They are commercially known as 40% strength low-freezing dynamite, 40% "straight" nitrogiycerin dynamite, 40% strength ammonia dynamite, and 40% strength gelatin dynamite.

The tests were carried out on the Monongahela River at Lock 1, Pittsburgh, Penn. The procedure followed was similar to that ordinarily employed in submarine excavating. A drill boat was moored over the rock by means of adjustable legs and by mooring lines attached to the shore, and the holes were drilled in the underlying rock, a hard shale, by 2½-in. steel drills guided by long iron pipes 4 in. in diameter. On Dec. 19, 1911, the depth of the water where the work was being conducted was 10 to 13 ft., depending on the elevation of the bottom of the river. The results of the tests showed that 40% "straight" nitroglycerin dynamite is the most suitable class of explosive for this submarine work.

If the work required longer intervals in the preparation of the shots, or if there were a material change in the character of the rock to be blasted, requiring explosives of different characteristics, the other classes of explosives might be used to advantage, subject to certain conditions.

The following table was prepared to compare the various classes and grades of explosives as to potential energy and disruptive and propulsive effects:

Class and grade.	Percentage strength representing potential energy	Average percentage strength representing disruptive effect	Average percentage strength representing propulsive effect.
30% "straight" nitroglycerin dynamite	93.1	84.1	96.8
40% "straight" nitroglycerin dynamite	100.0	100.0	100.0
50% "straight" nitroglycerin dynamite	111.0	109.2	107.4
60% "straight" nitroglycerin dynamite	104.0	119.8	114.9
60% strength low-freezing dynamite	60.2	93.5	91.2
40% strength ammonia dynamite	101.8	67.9	99.1
40% strength gelatin dynamite	105.7	78.4	95.8
5% granulated nitroglycerin powder	67.6	21.6	53.3
Black blasting powder	71.6	6.8	58.6

The relative disruptive and propulsive effects tabulated above are shown graphically in the accompanying figure.

These values are fairly consistent with general practice and should serve as a useful guide for comparing the practical value of explosives. It is worthy of note that the potential energy of 40% strength ammonia dynamite and of 40% strength gelatin dynamite (that is, the theoretical maximum work that these explosives can accomplish) is higher than that of 40% "straight" nitroglycerin dynamite, but that the disruptive and propulsive effects, which represent the useful work done as shown by actual tests, are less than those of 40% dynamite. Accordingly, 40% "straight" nitroglycerin dynamite is more economical for general use in blasting operations if the conditions and character of the work are such as to permit its use; nevertheless, the 40% strength ammonia dynamite and the 40% strength gelatin dynamite are more efficient and economical for certain kinds of work that require explosives having a large propulsive effect and a comparatively small disruptive effect.

For example, in blasting soft rock, 40% "straight" nitroglycerin dynamite, which has a very high percussive force, may be too quick in action, whereas the ammonia dynamite or the gelatin dynamite, having practically the same heaving and pushing action and less percussive force, will be more suitable. The tests also show that 60% strength low-freezing dynamite is not quite equivalent to the 40% "straight" nitroglycerin dynamite.

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Forest Products in 1912—According to the preliminary report recently issued by the U. S. Bureau of the Census there were 29,648 sawmills in operation in 1912, as compared with 28,107 in 1911. The total production of lumber, lath and shingles, in 1912 was 39,158,414 M. ft. b.m., an increase of about 6% over the figures for 1911, but a little less than the production for 1910. The average production per mill, however, was 5.3% greater in 1912 than in 1910. Quoting from the report:

Increases in production among the individual states were quite general, slight exceptions appearing in certain of the Eastern States and a few of the Western Mountain States, with, of course, the usual decrease in the output of the Lake States which has characterized their showing for several years, due directly to the rapidly decreasing supply of lumber material in this region. While both the principal lumber-producing centers, namely, the Southern States and the Pacific Coast States, reported larger cuts in 1912 than in the preceding year, the increased production in the first-named group was substantially greater than for the United States as a whole. The development of the lumber industry in the Southern States during recent years has been rapid. At the census of 1900, 38.7% of the total production in the United States was reported from this region, while in 1907 it contributed 45.7% of the output, and in 1912, 51.4%, or more than one-half of the total. The production reported from Washington in 1912 was the largest recorded since 1906. Although for nearly a decade this state has led all others in the production of lumber and shingles, in 1912 it contributed more than one-tenth of all the lumber and nearly two-thirds of the shingles manufactured in the United States.

Of the reported total lumber production, softwoods contributed 30,526,416 M. ft. b.m. More than nine-tenths of the present stand of yellow pine—the softwood which is drawn upon most heavily for lumber material—is in the yellow-pine belt, which comprises the Atlantic and Gulf Coast States from Virginia to Texas, inclusive, together with Missouri, Arkansas and Oklahoma. Under the term yellow pine are included the several species: Longleaf, shortleaf, loblolly, Cuban, etc. The reported cut from yellow-pine timber in this territory during the year amounted to 14,470,617 M. ft. b.m., or about 98% of the total output from this species in the United States. Douglas fir, the species which ranked next to yellow pine among the conifers or softwoods, supplied material for 5,175,123 M. ft. b.m. The production from both of these species was greater in 1912 than in the preceding year. White pine ranked third among the softwoods in 1912, though the cut from this wood was smaller than in the preceding year, and has been declining steadily for several years past.

The reported cut of hardwood lumber in 1912 was 8,631,998 M. ft. b. m. To this total oak, the leading hardwood species, contributed 3,318,952 M. ft., or 38.4%, and showed an increase over the output for the preceding year of 220,508 M. ft., or 7.1%. Maple, red gum, tulip poplar, chestnut, beech and birch followed oak in the order named.

Determining Power Possibilities on a Watershed

By Lyle A. Whitsit*

SYNOPSIS—An analysis of the power possibilities on a large watershed, supporting any number of power houses under one management. With these conditions, less stress is put upon uniform water flow than upon uniform power. An understanding of what may be expected under the latter requirement is best reached by the construction of a mass curve of energy of the water stored on the watershed. The methods of constructing this and the other curves, requisite to the power determination, are discussed.

Water-power developments have consisted, until recently, of single isolated power houses. Although there were other plants on the same stream, each power house was operated independently, without considering the effect of its operation upon the output of the others on the same watershed. In such cases the power plants were under separate ownership. But now, many entire watersheds are controlled by one interest, and it becomes necessary to determine the total maximum power possibilities of such a region, taking into account the effect of the operation of one power house upon the output of the others in the same system.

The problem involving one power house and its supplementary reservoir, is easily solved; but when any number of power houses and reservoirs must be considered, the problem is more complex. Especially is this so when there are some reservoirs which serve only for storage purposes. Cases where power houses are on different branches of the same river, and where the drainage areas are subject to different rainfall conditions, still further complicate the determination. When the periods of flow in the various river stretches connecting the power houses are different, and when some of the power houses are supplied directly from the reservoirs, and others through long penstocks or tunnels, the problem is one whose complete and exact solution demands careful attention.

When all the conditions affecting the power development are under the control of one management, and when all the suitable locations of reservoirs and power houses are given, together with their relative elevations, the object of the investigation is: (1) to determine the maximum power available, uniform over as long periods of time as possible, for all the previous years for which runoff data are accessible; (2) to analyze the ranges of power available at each power house; (3) to determine the installation required in each power house, the number and size of units, also the size of the power houses themselves.

A Common Method of Determination

The problem of determining the power possibilities is usually attacked along certain well known lines. In the case of a single development with no large storage facilities at hand, a hydrograph is constructed showing the amount of water which has flowed past the power house. It is extended over as long a period as there are stream data. From this, the minimum, which forms the basis of the later design, is determined.

*405 W. 118th St., New York City.

If the development is to contain a large reservoir, it is then possible to regulate the flow, and in the investigation a mass curve of the river discharge may be constructed for the runoff of the previous years. Such a curve is one in which any ordinate to the curve represents the total accumulated amount of water which has passed the station from the beginning of any initial time for which the data are available, up to the day represented by the ordinate in question.

Curve *B* (Fig. 1), for example, is a portion of such a mass curve. On Jan. 31, nine billion cubic feet of water have passed the station. It is seen that the tangent of the angle which any part of the curve makes with the horizontal axis is a measure of the river discharge, or the rate of flow, for that day. A line connecting any two points in the curve, represents the mean discharge of the river for the period between the two days indicated by the extremities of the line. For example, the mean discharge for the period from Jan. 15 to Mar. 11 is 4300 sec.-ft. (cu.ft. per sec.) of water.

This mean rate of discharge can only be maintained through a power house by the use of stored water. From Jan. 15 to Feb. 7, the natural flow of the river is less than the mean rate of 4300 sec.-ft. It, therefore, becomes necessary to draw upon the reservoir for water, in addition to the natural flow. The vertical distance from a point in the mass curve to the straight line, as *C*, represents the amount that has been drawn from the reservoir, up to Feb. 6. In order to maintain the uniform rate of 4300 sec.-ft., it is axiomatic that this distance must not be greater than the capacity of the reservoir for maximum draw down. It is also essential in investigating the advantages of a reservoir, that the mean rate line must either intersect or be tangent to some previous part of the mass curve as on Jan. 15; i.e., it should not be so constructed as to start from some point above the curve.

From Feb. 6 to 14, the natural flow is greater than the uniform draft, and the reservoir is gradually replenished. If the known runoff is represented by a mass curve, then for a given reservoir capacity, the water which was available during any period may be determined; after which, with a given acting head, the power may be computed.

An investigation over a long period is a tedious process and the writer has known some engineers to choose by inspection of the yearly hydrographs, an average year. The power available for this year is then determined, and used as a basis for preliminary consideration. The choosing of an average year, as will be shown later, is open to objection, on the score of inaccuracy.

By carrying out the above investigation the power from the regulated flow may be determined for any number of previous years for which data is at hand. It may be represented by a powergraph, which is a curve of power plotted to the abscissas of calendar time; or it may be represented by a time-flow curve, which indicates what percentage of the total period, any given amount of power was available. This time-flow curve shows the minimum power, and in like manner the percentage of time

that power, equal to or greater than any given amount, was available, also its amount for any given percentage of the time.

The choice of an average year and the basing of conclusions upon the results derived therefrom, is not advisable except for preliminary investigation. The selection may or may not be a fortunate one. The year giving the average runoff, does not necessarily give the average power for the period, especially where the development contains reservoirs. The distribution of the rainfall throughout the rainfall year exerts a large influence on the results obtained. The amount of power available is the information desired and not the runoff.

The conditions should be studied for as far back as there are authentic data.

After a preliminary analysis, a given set of rules may be assumed as governing the operation of the future power system. A further investigation should be made to determine whether they could have been followed in the previous years for which water data are available.

This method is suitable for determining the power possibilities of a given development, when one or two power houses with accompanying reservoirs are involved. From the mass curve of runoff-discharge, the available water for power is obtained, and this, under given net acting heads, will determine the power available.

FIG. 2 - DRAINAGE MAP SHOWING LOCATIONS OF THE RESERVOIRS AND POWER HOUSES

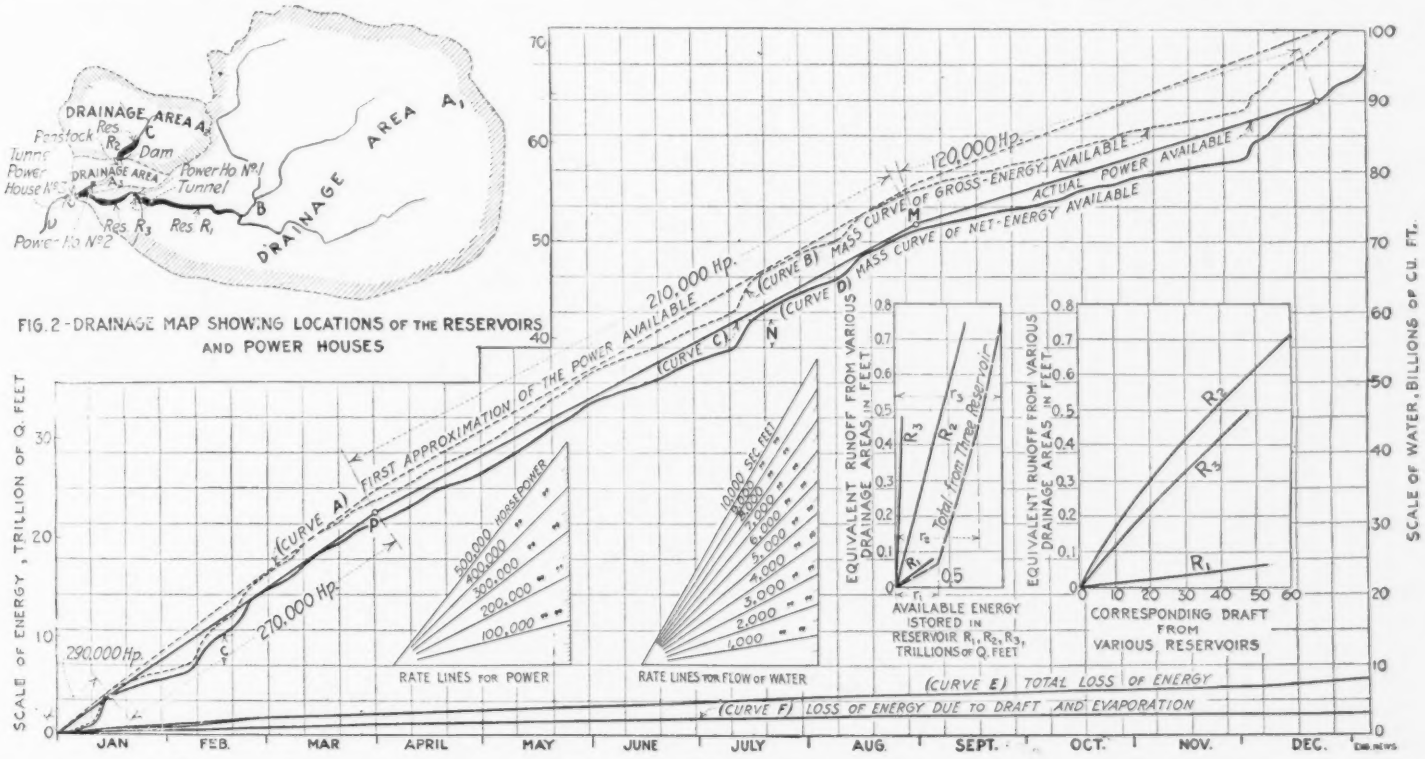

FIG. 1. CURVES FOR DETERMINING THE POWER POSSIBILITIES ON A LARGE WATERSHED

The ranges of available power over a long term of years, is of much greater value than over an average year. The investigation of an average year is unduly influenced by the beginning and the end of the calendar year; i.e., the rainfall conditions at the end of the previous year and the beginning of the following year may have an important influence on the results obtained. It is not enough to begin with the reservoir full and leave it full at the end of the year. In actual operation no attention is paid to the calendar year.

A hydraulic year (the period of time during which the rainfall conditions are repeated) cannot be chosen to give an average for a number of calendar years; for they are, in themselves, of different length, and need not be equal in length, to a calendar year. One is, therefore, not justified in choosing one calendar year to represent an average for a long period, either as to minimum or maximum power, or the distribution of power throughout the period.

In certain electro-chemical processes the power which is available for only a part of the time, is of value; and its correct determination as to amount and duration becomes important. The investment for which the investigation is made is on the basis of a long term of years, and an average year may not give these data correctly.

When a large number of related power houses and reservoirs are involved, this method of using the mass curve of discharge becomes very long and tedious. Also, it is only approximate, as it gives as a result uniform flow of water, not uniform power. It fails to take into account the regulative effect on the *power output* of the power houses situated on the upper section of the watershed.

A Proposed Method of Determination

In carrying out a set of investigations for a development involving some fourteen reservoirs and eight power houses, all on one watershed and all to be operated in one connected system, the writer has devised a method which gives the total power available for the entire system. It requires the construction and use of an accumulative curve, for which the term, "Mass Curve of Energy," has been adopted.

CONSTRUCTION OF THE MASS CURVE OF ENERGY—In the case of the mass curve of discharge, any ordinate to the curve was a measure of the total discharge of the stream up to the time in question. In like manner, any ordinate of the mass curve of energy is a measure of the total available energy in the runoff from the watershed, from any starting time up to the time represented by the ordinate. The value of this ordinate involves both the

amount of water passing through each power house and also the acting head through which the water acts at each power house.

For example, the ordinate to Curve *B* (Fig. 1) indicates that, from Jan. 1 to Mar. 14, there was an amount of energy available equivalent to 20 trillion cu.ft. of water under a one-foot head. The writer has adopted as a unit of energy, the Q.ft., analogous to the term of sec.-ft. as used for the measure of discharge. The symbol is an abbreviation for cubic feet of water under one-foot head.

Here again, as previously stated, the tangent of the angle between a tangent to the curve and the horizontal, is a measure of the rate at which the potential energy is used, or the amount of power. It should be noted that the power is obtained without further computation.

For the sake of convenience a scale of rate or power lines is drawn adjacent to the mass curve. The slopes of the tangents on the mass curve, when referenced to this scale, will give the gross power available for the periods in question. A straight line joining any two points on the curve, as *PM* on Curve *C*, when referred to the rate lines, indicates the uniform power which may be mantained over this period, providing there is sufficient reservoir capacity to tide over the deficiency equal to the ordinate *N*, on July 10.

In like manner a mass curve of energy may be constructed which will give the total available energy over an entire rivershed. Let x_1, x_2, x_3, etc., represent the runoff in cubic feet, at the various entrances to penstocks, or tunnels, of a system of power houses. This runoff may be read from daily gage heights at a rating station; or, for preliminary purposes, it may be taken as a certain proportion of the runoff at a government station, on the river in question.

Let h_1, h_2, h_3, etc., represent respectively the gross acting head at each power house, with reservoirs full. In the case of power houses supplied through long penstocks, the loss of head in the penstock line varies with the amount of water used. In preliminary investigations, a mean value can be taken for all conditions; or, as will be shown later, the total head from forebay to tail race may be used, and correction applied for the loss of energy due to draft, evaporation, friction in penstocks, head races, etc.

Then if x is the discharge for the day, the total energy available from the natural flow of the river for that day is $x_1h_1 + x_2h_2 + x_3h_3 +$ etc., and $\Sigma\, x_1h_1 + \Sigma\, x_2h_2 + \Sigma\, x_3h_3 +$ etc., is the total energy available over any given period. Often for approximate results, x_1, x_2, etc., can be expressed in terms of the daily runoff at one or two government stations on the river. In this case the expression becomes $x\,(c_1h_1 + c_2h_2 + c_3h_3 +$ etc.), where $c_1h_1 + c_2h_2 + c_3h_3 +$ etc., equals a constant, and x the daily runoff at the government station.

Curve *B* (Fig. 1) represents the mass curve of energy for a development shown on the map in Fig. 2. The system of development as here assumed for the purpose of illustration, contains three reservoirs R_1, R_2 and R_3. The first two of these are located on different tributaries of one river, and the last is situated on the main river below the intersection of the two tributaries. The first is supplied by water from reservoir R_1, through a short tunnel; the second supplied from reservoir R_2, through a long penstock, and the third from reservoir R_3, through a tunnel.

Curve *B* represents the mass curve of *gross* energy available at these three power houses for one year. Upon this curve, rate lines may be superimposed which will represent the total gross uniform power available from the assumed system.

Construction of the Reservoir Capacity Curve

Before establishing the rate lines, it is necessary to know the amount of energy stored in the various reservoirs, which may be called upon to produce a uniform power from the irregular flow of the river. The energy available from the water in any one reservoir, is equal to the amount of water in the reservoir multiplied by the sum of the acting heads through which this water is used. The water from storage near the headwaters of a stream passes through all of the lower power houses. In computing the value of the total head through which the water acts, due consideration should be given to the fact that all the reservoirs will be drawn upon simultaneously, thereby gradually decreasing the head. Hence the water first drawn from the reservoir is used through a much greater head than that which is taken out last.

Before determining this storage, a method should be adopted for drawing on the reservoirs. There are two schemes to choose from, one by which the resulting loss in energy due to loss of head from reservoir draft is the least; and the other one, by which there will be the least possibility of losing water, if during a subsequent heavy rainfall, the reservoirs fill up and spill over, thereby losing power in the water which does not pass through the turbines. By the first method, that reservoir would be first drawn down completely whose water was used through the highest combination of acting heads. The reservoir giving the next highest energy per foot of draft would be drawn upon. This method would be used only when it was known that an extended low-water period was following, and when it was known that the combined storage would be used before the next rains replenished the reservoirs. It is the more economic procedure; but it cannot be used in the early part of the year, when there is only a temporary falling off of the water-supply.

Should heavy rains come while one reservoir is depleted, and the others are filled, the runoff draining into the filled reservoirs would be nearly all wasted. Accordingly, it will be advisable to assume that each reservoir is drawn down an amount which represents a uniform depth of water over its own contributing drainage area. To illustrate: if reservoir R_1 (Fig. 2) had a capacity which would be filled by a runoff of 0.1 ft., over the drainage area A_1, directly tributary to this reservoir; reservoir R_3, a capacity equivalent to 0.4 ft. over its drainage area, A_3 and reservoir R_2, a capacity equivalent to 0.5 ft. over area A_2; then the first rate of draft would be an amount from each reservoir equivalent to an equal runoff from its contributing drainage area. This rate would be continued until reservoir R_1 had been depleted. If, before this time, a heavy rain should come, all of the reservoirs would be filled simultaneously, and there would be no waste of water from one full reservoir, while the others were recuperating.

It would be well to arrange, if possible, the reservoir locations and capacities so that the ratio of capacity to contributing drainage area, for the ultimate development,

would be the same for all the reservoirs. Such a system based on a uniform, simultaneous rainfall and runoff, over the watershed, would give an ideal system for operation. This, of course, can scarcely be realized; the locations of reservoir dams being dependent upon suitable dam sites and suitable locations for power houses to obtain the necessary pressure heads. This purely theoretical consideration should be kept in mind in the design of a comprehensive system.

In the above assumed case, reservoir R_1 will be depleted first. The further rate of draft should now be such, that if a heavy rain comes, R_2 and R_3 will be filled simultaneously. In the latter case R_1 will be only partially refilled. If the rain continues beyond this time, reservoir R_1 will continue to refill; but water will be wasted from reservoirs R_2 and R_3, unless the tunnels, penstocks and installations are sufficient in power houses No. 2 and No. 3 to take care of the flood waters, a condition which would not ordinarily be provided for.

The fundamental requirement for the operation of such a system would be, of course, to waste as little water as possible. This principle is to be carried out as is seen, by drawing on the reservoirs in such a way that the excess capacity of one will bear the same relation to any other as do the corresponding drainage areas, directly tributary to the respective reservoirs. It should be borne in mind that this area is the area of the rivershed between the two adjacent dams and that it is not the total drainage area above the reservoir in question.

In determining the amount of available energy in a reservoir system, a hypothetical case will be assumed. Fig. 2 represents the outline of a watershed, in which two rivers, B and C, unite to form a third D. Power houses are to be located at No. 1, No. 2 and No. 3, with the supplementary reservoirs R_1, R_2 and R_3.

In an investigation of this nature, the future development should be kept in mind, even though the power houses and reservoirs farther up the river, are not to be built for an indefinite period.

An energy curve should be constructed for this system of power houses and reservoirs. The total energy available at any given time depends upon the water available, and upon the net acting head at each power house. The latter at each point is equal to the gross pressure head minus the loss of head due to draft, evaporation, and the frictional losses in the head race, tunnels, penstocks, etc. The draft depends upon the amount of water which has been drawn from the reservoir storage; and the frictional losses are proportional to the amount of water which is being drawn through the system at the period in question.

To determine these controlling factors it is necessary to make a first approximation of the power developed. The mass curve of energy is constructed on the assumption that the total gross head is available at each power house, and serves a temporary purpose; for from it, the first approximation of the amount of power available is determined. From this latter, the losses of head due to draft and friction may be computed. It is now possible to construct the correct mass curve, representing the actual amount of power or energy available. Upon this latter curve the rate lines of uniform power may be drawn.

Curve B (Fig. 1) represents a mass curve of gross energy for the above watershed, where the gross head is assumed to be available at the entrances of each of the three power houses. Upon this curve the rate lines. Curve A, indicating the first approximations of the uniform power, are drawn; care being taken to see that that portion of the ordinates between the mass curve and the rate line is not greater than the energy stored in the reservoirs. In fact, on the first approximation, this should be somewhat less than the maximum energy available.

The energy that is available in the reservoir system is shown by the curve entitled, "Available Energy Stored in Reservoirs R_1, R_2 and R_3." The abscissas of the storage curve are trillions of Q.ft. The ordinate axis of this curve represents the equivalent amount of runoff from the drainage areas, directly tributary to each reservoir, in feet of water. The reservoir capacity R_1 is equivalent to 0.067 ft. of water runoff from the drainage area A_1; the reservoir capacity of R_2 is equivalent to 0.756 ft. of water, on drainage area A_2; and the reservoir capacity R_3 is equivalent to 0.488 ft. of water, on the drainage area A_3.

When a draft of water equivalent to 0.067 ft. over each tributary drainage area, is drawn from each reservoir, the total energy available, in this system of power houses, is represented by r_1, and the entire amount of water available from R_1 has been used. When an additional draft of water has been drawn from reservoirs R_2 and R_3, above that which has already been drawn from these same reservoirs, equivalent to an amount of 0.421 ft. $(= 0.488 - 0.067)$ over the drainage areas tributary to R_2 and R_3, the total energy drawn from storage is represented by the abscissa r_2, and the entire amount of water available from R_3 has been used. There still remains some water available in reservoir R_2, which is equivalent to 0.268 ft. of water on the drainage area directly tributary to reservoir R_2. Should this be used, the total available energy has been utilized and is represented by the abscissa r_3. Care should be taken in computing these values, to make the proper deductions for the losses of head due to draft and friction.

In the investigation of either mass curve, the ordinate between the rate lines and mass curve should not be greater than r_3. Another point to be kept in mind is that these values, as well as those of the ordinates of the mass curves, involve the efficiency of the turbine and generator equipments.

There is another energy loss, requiring a more detailed explanation. As an example of this, let us consider the period of flow for Feb. 6 to 24. On Feb. 6 there is a draft of a value c, required from the reservoir energy. The storage curve shows that this requires an amount of water equivalent to 0.067 ft. on the drainage area A_1, and 0.270 ft. on drainage areas A_2 and A_3. Reservoir R_1 is depleted; and reservoirs R_2 and R_3 are only partially drawn down, when the maximum draft required has been reached to tide over the temporary decrease of the runoff. But on Feb. 6 the rate of rainfall has increased, resulting in a greater runoff than is required for developing power; the reservoirs then start to fill.

Assuming a uniform rainfall, it is evident that reservoir R_1 will be filled and will overflow before R_2 and R_3 are completely replenished. All the water overflowing from reservoir R_1, not passing through the power house No. 1, will be wasted; and a proper deduction should be made in the mass curve, following this date, to represent

the lost energy in the water wasted, and this value added to that represented by Curve F.

In the actual operation of such a system, as soon as reservoir R_1 started to fill at such a rate as to be filled in advance of R_2 and R_3, the operator would cease to draw upon the water in the latter two, and open power house No. 1 to its full installed capacity. But, even with such precautions it will not always be possible to conserve all the water.

It should also be taken into consideration, in this system, that the overflow from R_1 passes into R_3, and after R_1 has been filled, will accelerate the filling of the other.

From a study of the curves A and B, curve F may be constructed, whose ordinates represent the loss of energy due to the daily draft and evaporation, existing in the various reservoirs. Upon this, the curve E is superimposed so that the ordinates between curves E and F, represent the lost energy due to friction in the water courses. The total ordinate to curve E represents the total loss.

A second mass curve D is now constructed, any ordinate of which for any given day, is equal to the ordinate of curve B for that day (the gross amount of energy) minus the ordinate of curve E for that same day (the total loss of energy). This new curve represents the energy available for power. An adjusted set of rate lines is now superimposed upon this mass curve, again noting that any required reservoir capacity is not greater than the capacity in the reservoir system. The power indicated by the final rate lines is the value required.

This analysis should be continued over the entire period for which there is data. The power for the long term may be indicated by a powergraph or as a time-power curve. The powergraph, a curve used extensively to represent graphically the power or water history, shows clearly the maximum and minimum power and its duration. It also shows the favorable and unfavorable years, and is of great value in designing proposed installations.

The time-power curve indicates the percentage of the total period of time a given amount of power was available. In electrical manufacturing processes, it is often a commercial and economical proposition to allow a portion of the equipment to lie idle for a small percentage of the time, in order to have sufficient equipment to make use of the excess power available for the balance of the time.

Conclusion

The rainfall and runoff were assumed to be uniform over the entire watershed, although such may not be the case; but for lack of better information, it serves as a rational basis for the investigation of power possibilities.

When a given development is being considered, surveys are being made and river gages are being established at substations, it is also advisable to establish rainfall gages at as many critical points as possible, that the relation betwen rainfall and runoff, together with the sequence of these relations, may be established for individual sections of the watershed.

After sufficient data of this nature have been accumulated, the investigation may be carried further, and a set of rules derived for the operation of the complete development.

The installation of the rainfall and river gages should be continued and reports of rainfall, water stages, and reservoir storage telephoned to the superintendent of the system. His function is partly analogous to that of the train dispatcher of a railroad, or the load dispatcher of a great central-station system. From the conditions reported, he transmits orders to the several station operators, indicating the quantity of water they may draw, and the power which should be developed.

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Erection of a Cable Lift Bridge

By W. J. Howard*

An interesting bridge is now under erection to carry the four tracks of the Pittsburg, Fort Wayne & Chicago Ry. across the Calumet River at South Chicago, Ill., and will replace a double-track swing bridge. The structure consists of two double-track vertical-lift cable-operated bridges of the Waddell & Harrington type; they are placed close together but operate entirely independently. They are 210 ft. long, crossing the stream at a skew of 50°, and giving a channel 140 ft. wide. The clear headway for shipping is 120 ft., and the spans are being erected on falsework at this elevation, an opening 80 ft. wide being left to enable vessels to pass.

The east tower of the north span was erected with a guy derrick designed specially for this work. It has an 85-ft. mast and a 70-ft. boom. The mast is built of two 24-in. 80-lb. I-beams, with an 18-in. 55-lb. I-beam placed horizontally between their webs as a separator, and with lacing over the flanges of the 24-in. beams. The boom is built of a pair of 18-in. 55-lb. I-beams, with lacing over the flanges. There are eight guys of 1¼-in. plow steel rope, and the spread of the guys is only 16 ft. from the foot of the mast. This is shown in Fig. 1.

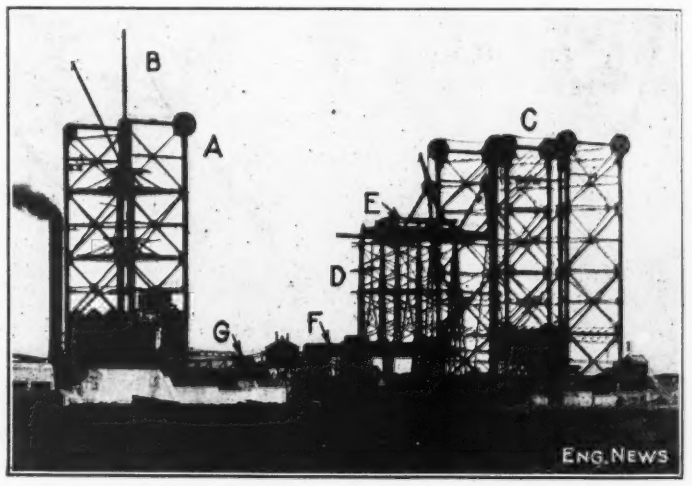

Fig. 1. Towers and Falsework for the Cable-Lift Drawbridge over the Calumet River at South Chicago; Pennsylvania Lines

A. Tower for one lift span.
B. Guyed derrick erecting the tower.
C. Towers for two lift spans.
D. Falsework for erecting trusses.
E. Traveler for erecting trusses.
F. Girders and chord sections forming an open space in the falsework, to clear a swing bridge.
G. Swing bridge, whose end swings into opening left in falsework.

After the tower was completed, the adjacent east tower of the south span was erected by a 68-ft. trussed wood boom, 16x16 in. One of the tower posts of the south span was utilized as a mast, the boom-seat and topping-

*Engineer, Kelly-Atkinson Co., Chicago.

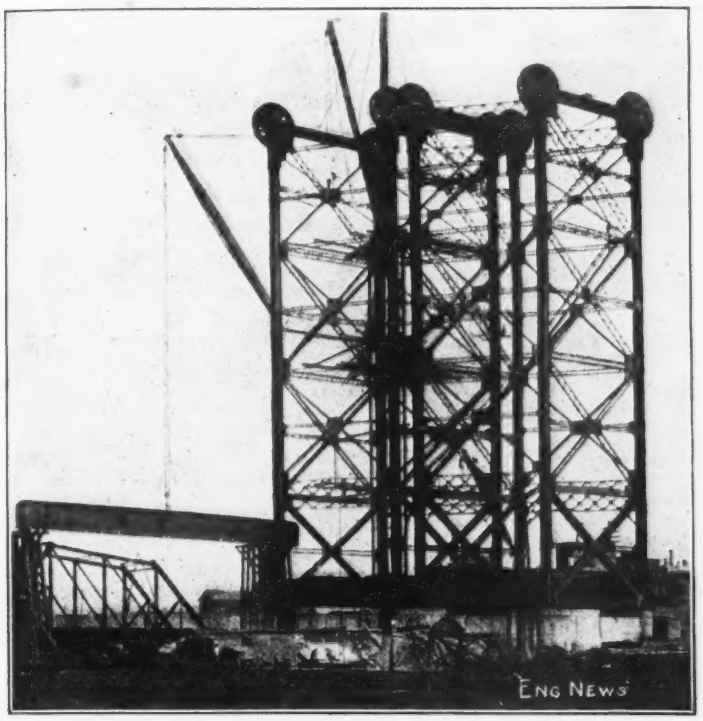

FIG. 2. DERRICK BOOM UTILIZING A TOWER POST OF THE CABLE-LIFT BRIDGE AS A MAST

(The boom is placing one of the 103-ft. girders to form an opening in the falsework to clear the end of the swing bridge.)

lift brackets being of special design and riveted to the tower post. This boom is shown in Fig. 2, with the guy derrick above it.

The general design of the falsework for the truss erection is shown in Fig. 3. The swing bridge of the Lake Shore & Michigan Southern Ry. is parallel with and only 25 ft. from the lift bridge, and its end swings under the north span of the latter. It was necessary to build the falsework so as to avoid interfering with this bridge. This was done by placing in the falsework two 103-ft. girders (supported by timber bents) on the side nearest the swing bridge, and three top-chord lengths of the span side by side (also on timber bents) on the side farthest from that bridge. Fig. 1 shows the girders and chord sections in place, side by side. These were all at an elevation of 40 ft. above the water, which was sufficient to clear the swing bridge when open. Upon these temporary steel spans was erected the upper part of the falsework for the trusses. Fig. 3 shows the longer side of the knee-brace falsework.

On the west side of the river, a timber knee-brace support resting on the pier and inclined outwards over the channel will support two panels of the truss (52 ft.) on the longer side of the skew and one panel (26 ft.) on the shorter side. The bottom chord connections of two of the 70-ft. end sections of the chords will come over the 80-ft. ship channel which is kept open for navigation, and the cantilever ends of the chords will be supported by a mule traveler on the main falsework while the connection is being made across the channel. Cables slung across the river from the 15-ton sheaves on the towers will help to support the chords till the truss is completed.

The material for the bridges was built by the Pennsylvania Steel Co., and the inspection is in charge of the Pittsburgh Testing Laboratory. The Dravo Contracting Co., of Pittsburgh, had the contract for the substructure. T. M. Bole is division engineer in charge for the Pennsylvania Lines. The writer is engineer for the Kelly-Atkinson Co., of Chicago, which has the contract for the erection. Work on the erection was commenced on Dec. 10, 1912, and is expected to be completed about Sept. 15, 1913. The erecting force averages 30 men. The weight of steel is about 550 tons in each of the four towers, and 1100 tons in each of the two spans The heaviest- single pieces handled weighed about 30 tons.

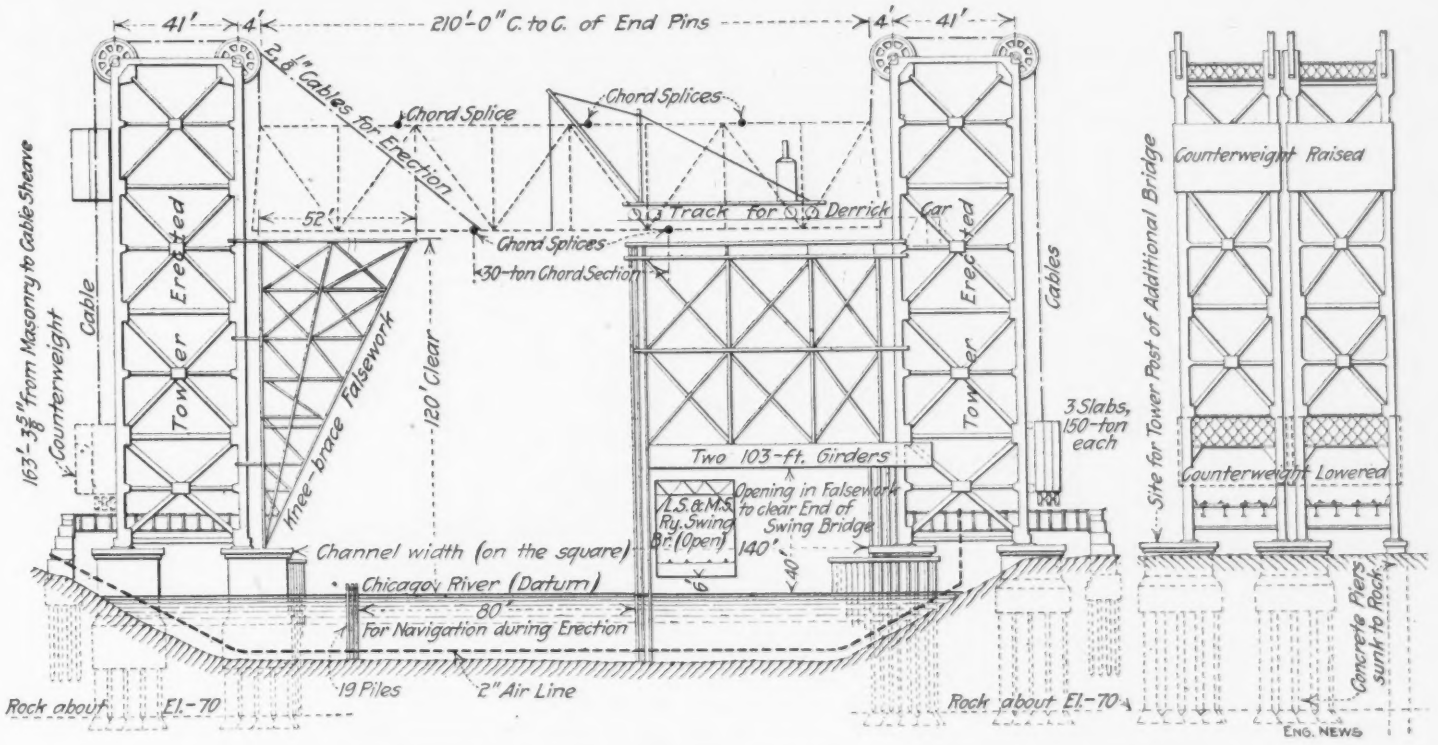

FIG. 3. METHOD OF ERECTING THE 210-FT. TRUSSES FOR THE CALUMET RIVER LIFT BRIDGE OF THE PENNSYLVANIA LINES

A Study of Irrigation Heads in the Modesto and Turlock Irrigation Districts, California

The size of "head" (volume of water in cu.ft. per sec. delivered to each individual irrigator) and the length of time the head is to be used per acre of land are a vital factor in the design and operation of irrigation works. To secure definite information based on actual practice, Edwin Duryea, Jr.,* sent an assistant engineer to make a study of actual practice in the Modesto and Turlock irrigation districts. This study was made near the close of the irrigation season of 1912 (observations in late July; report dated Aug. 8, 1912), by P. C. Berkefeldt.†

Mr. Duryea has kindly sent us a copy of Mr. Berkefeldt's report, from which we have taken the following statements:

The Modesto Irrigation District is located in Stanislaus Co., is bounded by the Stanislaus, San Joaquin and Tuolumne rivers, and has a total area of 81,143 acres, of which 40,000 acres are under irrigation. The Turlock Irrigation District lies south of the Modesto, in Stanislaus and Merced counties, is bounded on the south by the Merced River, and has an area of 176,210 acres, of which about 80,000 are under irrigation. Each district takes its water-supply from the Tuolumne River, through joint headworks near the boundary of Stanislaus and Tuolumne counties. The soil in each district is for the most part sandy, that in the Modesto District ranging from "almost pure sand to a fairly stiff clayey, sandy soil," and that in the Turlock District being even more sandy, but with spots of "almost pure sand" alongside of comparatively "stiff clayey, sandy loam." In the Modesto District there is a tract of some 1800 acres of "heavy, red, gravelly soil." In general, the depth to hardpan in each district is from 3 to 12 ft., but in the 1800-acre tract just mentioned it is only from 1½ to 3 ft. deep.

Alfalfa is the principal crop raised in each district, its acreage being 86.4% of the total area under irrigation in the Modesto and 70.8% in the Turlock District (in 1912).

The Three Systems of Checking

For purposes of water distribution the land is divided or checked in accordance with three different methods, known as the contour-check, the square-check, and the strip-check. These methods are described by Mr. Berkefeldt as follows:

THE CONTOUR CHECK—The contour system was the first used for checking ground. In it the shape of the check follows the contours to a greater or less extent. Low levees or banks are built along the contours and the ground leveled between them. The ground is either leveled parallel to the contours as in the smaller checks, or is given a slight slope in that direction.

The great objection to this system is that it gives checks that are very irregular in shape, which makes it more difficult to work the land than if they were of a more even shape. For this reason whenever it becomes necessary to recheck ground, either the square- or strip-check system is used.

The width of the checks vary greatly, from about 30 ft. to 100 ft. Some maintain the same width throughout the check, while others taper down to a point on one end. The length of the checks vary from about 100 ft. to about 600 ft. The grade varies from about level for some of the smaller checks, to a maximum of about 0.5 to 1 ft. in 100 ft. in the longer checks.

SQUARE-CHECK SYSTEM—In this system the ground is graded into square or rectangular shaped pieces varying in size from about ½ acre to 3 acres. The best size is considered to be about 1 to 1¼ acres. Some irrigators prefer to have the grade of their checks about level, while others prefer as much as 0.5 ft. grade in the length of the check. The best grade for a 1- to 1¼-acre check is about 2 to 4 in. in the length of the check.

The square-check system is considered by some of the irrigators, especially those that have their land already checked up by this method, to be the best system. However, some of the irrigators who use the square-check system, think the strip-check system superior to the square-check.

STRIP-CHECK SYSTEM—In this system the land is divided up into long narrow strips, the width varying from about 30 to 100 ft., and the length from 150 ft. to about a half a mile. The best width is considered to be from 50 to about 75 ft. The best length depends on the amount of land an irrigator owns, varying from about 300 to about 1320 ft. The grades used vary from about level for some of the smaller checks to a maximum of about 1 ft. in 100 in some of the longer checks. The best grade is considered to be from about 2 to 4 in. in 100 ft.

The strip-check system is considered by most irrigators to be the best system for checking land. It is by far the cheaper to install and is more economical of water.

Heads, Check Systems and Other Practices in the Modesto Irrigation District

Each district is taken up separately by Mr. Berkefeldt. We quote his remarks on the Modesto District nearly in full:

HEADS AND TIME—The head during the present (1912) irrigation season up to July, has varied from about 16 to 20 cu.ft. per sec. The time limit is a half hour per acre. This means from 8 to 10 in. per irrigation. The irrigators think that they are getting from 4 to 6 in. per irrigation. The Superintendent of Ditches stated that a head of 15 to 20 sec.-ft. for 20 minutes would give an irrigation of from 4 to 6 in.

The District recently passed an order fixing the minimum head at 15 sec.-ft. and time limit to 20 minutes to the acre.

From July 23 to 25, when the water was running low, and before it was bunched, both the irrigators and ditch tenders greatly overestimated the amount of water they were using; in many instances their estimates were twice as large as given by the meter gagings. In only one instance was the estimated head smaller than the result obtained by the meter gaging.*

The conditions at this time were not normal, and the irrigators were greatly discontented, for on account of the time limit of 20 minutes to the acre, and the small head they were using, they could either irrigate part of their land well, or else all of it with a comparatively light irrigation.

With an actual head of 5.6 sec.-ft. irrigating one 1¼-acre check, sandy soil, crop alfalfa, the water had traveled about two-thirds of the length of the check in 55 minutes, when it had to be turned off. Toward the end the water was sinking into the ground about as rapidly as it was supplied to the check, without gaining very much headway along the check.

With an actual head of 12.7 sec.-ft. irrigating sandy soil, crop alfalfa, the time averaged about 76 minutes to the acre. This means an irrigation of about 15 in. depth. The irrigator estimated this as from 4 to 6 in. depth. In this particular instance the irrigator will only be able to irrigate part of his land.

With an actual head of 7.1 sec.-ft. irrigating one acre fairly sandy soil, crop alfalfa, the time for a fair irrigation (depth estimated by irrigator from 4 to 6 in.), averaged about 50 minutes. This means an irrigation of 5.8 in.

For fairly sandy soil and sandy soil, crop alfalfa, it is considered best by both irrigators and ditch-tenders to use a larger head for a shorter length of time; the general opinion is about from 15 to 20 sec.-ft., but preferably between 18 to 20 sec.-ft., and a time limit of 20 minutes to a half hour per acre, covering the ground to a depth (estimated by them) about 4 to 6 in. This is an actual irrigation of from 6 to 10 in.

With a smaller head than about 15 sec.-ft. if the check is large or if the soil is very sandy, or if the ground is not checked up well (i.e., the grade too flat and ground uneven,

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*Of Duryea, Haehl & Gilman, Civil and Mining Engineers, Humboldt Bank Building, San Francisco, Calif.

†Assistant Engineer, South San Joaquin Irrigation District. The report was made for the benefit of the district just named, Mr. Duryea being chief engineer of that district.

*Mr. Berkefeldt measured the irrigation heads by means of a Lallie current meter, using the two-point system (⅕ and ⅘ depth).

or if it happens to be a high check with respect to the ditch it is to be irrigated from), it takes too much water and too much time to cover the land.

It is considered best to flood the check quickly, and not allow the water to stand on it for any length of time. Wherever possible the best way is to shut the water off before it has quite reached the lower end of the check, gaging the water so that it will just cover the lower end to the proper depth with the excess water at the upper end of the check. This, of course, can only be determined by repeated trials, but the general practice is to give the land all the water they can with the head and time limit they have.

The dimensions of the farmers' ditches vary from about 4 to about 8 ft. on the bottom. The side slopes vary from 1½ to 1 to to 2 to 1. It is considered best not to have the ditches less than 6 ft. on the bottom. The gates vary from 3 to 8 ft. in width. It is considered best to have the gates not less than 6 ft. Both the gates and frames are constructed of wood. The average life of these structures is claimed to be from three to four years.

Checks are used in the canals to back up the water and gain head; in some places the checks are closer than half a mile.

They use rotation, size of areas being about 3000 acres, averaging one irrigation in from 30 to 35 days.

It is claimed that one man under ordinary conditions can handle a head of from 10 to 40 sec.-ft.; however, this has never been accurately measured.

In the District they try to avoid all fluming. Wherever possible, they keep their subgrade below ground surface, but if this is not possible a distance of 1 to 1½ ft. above ground surface is never exceeded.

The general opinion against fluming is very strong. In lateral No. 2 the District has tried out some McGinnis metal flumes on concrete abutments and found them to give very satisfactory results, the cost of maintenance being very low in comparison to the wooden flumes.

The reservoir with a capacity of about 30,000 acre-feet is used more as a regulator for the flow of water than for storage purposes. Last year, on account of failure of the flumes above it, it saved about two weeks' irrigation.

The reservoir cost $250,000, or about $8.33 per acre-ft. It is thought worth while to pay from $15 to $25 per acre-ft. for storage.

Last year the water tax varied from about $1.68 per acre for outlying land to as much as $4 per acre for some of the land near the town of Modesto. The tax this year is expected to be higher.

The capacity of the Main Canal is 735 sec.-ft.

For the irrigation of trees, vines, etc., only about 4 to 6 sec.-ft. are required. The water in this case is allowed to run down the rows, about three rows being irrigated at once.

Most of the irrigators are on their fourth run (7-25-12), which they will complete; this will be the last for the season. Some have already started on the fifth, which they will not complete.

In the northeastern section of the District, in the hard land section, where the land is a heavy, red, gravelly soil and the depth to hardpan runs from about 18 to 30 in., they are working on their seventh run. This ground requires a larger number of irrigations with less depth of water per irrigation. The average depth of an irrgation is about 2½ in. As far as the amount of water is concerned, this soil takes about as much as the sandier soils, but at different intervals of time.

There are no records of any meter gagings having been taken in the District. The heads are measured over weirs and the volumes computed according to overpour and pressure tables.

Some of the ditch tenders claim that this method is sometimes inaccurate on account of timbers which hold the weir in place or form part of the head gate, etc., which tend to obstruct the flow of the water and give a higher gage reading than would occur in the case of free flow.

Practice in the Turlock Irrigation District

The differences in practice in the two districts are not sufficient to warrant giving much space here to the section of Mr. Berkefeldt's report which deals with the Turlock District. A few points will be noted.

The head prior to 1912 was 12 sec.-ft. (called an alfalfa head) and the time one hour per acre. A larger head having been found advantageous, the head was increased in 1912 to 20 sec.-ft. at the intake, giving an average head of about 17 sec.-ft. The time was decreased to one-half hour per acre. In the latter part of July, 1912, "the water was bunched in order to maintain a full head and the time limit" cut to 20 minutes. The local estimates of heads were found to be much closer to Mr. Berkefeldt's meter readings than were the estimates in the Modesto District. Meter gagings had been taken locally on the main canal and on the main laterals, and apparently rating curves had been established.

The irrigators use rotation, on areas of from 1000 to 6000 acres, with an average of 30 to 35 days between irrigations. Under average conditions one man handles a head of about 20 sec.-ft. for alfalfa and 4 to 6 ft. for trees, vines, etc.

General Conclusions

The general conclusions drawn by Mr. Berkefeldt from his studies of both the Modesto and Turlock Districts are as follows:

With regards to the different systems of checking, the strip system is best wherever the topography will allow it. It is also the cheapest way, and is most economical of water. Where this cannot be put in a combination of square-checks and strip-checks may be used to advantage. The best width for a strip-check is from 50 to 75 ft. The best length seems to be a point of dispute, depending on the size of a man's land, probably about 600 ft.: the best grade from 2 to 4 in. per 100 ft.

The general opinion with regard to farmers' ditches is that they should be at least 6 ft. wide, and the gates should be of a similar width.

For fairly sandy soil and sandy soil, crop alfalfa, it is better to use a larger head for a shorter period of time. For the average run of the land to be irrigated (i.e., the average grade, length, and width of checks, etc.), it is best to use not under a 15-sec.-ft. head for a time of about 20 minutes to a half hour per acre; depth of irrigation from 4 to 6 in., irrigating one check at a time.

The quicker a piece of land is irrigated and the less time the water is allowed to stand on the land, the better are the results.

In fairly sandy soil, the irrigation of trees, vines, etc., about a 5-sec.-ft. head is best, irrigating from three to four rows at once; time about one hour to an acre.

A large saving of volume of water necessary for good irrigation can be made by having the land well checked, having proper grade, width and length of check, etc.

In all cases where possible, the elevation of the water surface of the ditch should be at least 1 ft. above the highest check.

In all cases the water should be applied to each check directly from the ditch and should not be allowed to flow from one check to another, as the latter is very wasteful of water.

The estimates of the different irrigators with respect to the depth of water used per irrigation were very erroneous, and much lower than the actual amount of water they were using.

In general, the irrigators use as much water as they can possibly get on each irrigation.

The districts seem to have no methods of measuring the water and the size of heads vary widely. There is some talk among the irrigators of having accurate measurements made and have each irrigator pay for the total volume of water each uses rather than by a tax of a certain amount per acre.

The sentiment is very strong against wooden flumes on account of the high cost of maintenance, and wherever they can be avoided, this is done. As fast as possible they are replacing them by hydraulic fills with lined canals.

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Railway Tie Purchases in New England—During the past year the New York, New Haven & Hartford R.R. and its subsidiary electric lines purchased 2,904,640 cross-ties, costing nearly $1,200,000. The railway alone bought 2,376,000 ties, all but about 300,000 of which were made from native New England woods. Rejected ties bought for fence posts cost $100,000 more. The greater part of these ties were purchased of small farmers, payments running in amounts from $12 to $10,000. The railway company's specifications and prices are posted in all the railway stations and any number of ties offered will be taken. The New Haven system uses ties only of oak, chestnut or cedar. The larger number of the ties were purchased in western Massachusetts and Connecticut, where a tie train often picked up 2000 ties a single day.

Difficult Tunnel Work on the Metropolitan Railway of Paris, France

The construction of the underground system of the Metropolitan Railway of Paris has involved many special features and many difficulties, and some of the works have been described in our columns. In the present article we give particulars of some very exceptional examples of tunnel construction, abstracted from a paper by Mr. Godfernaux in the *Revue Generale des Chemins de Fer.*

Tunnels through Underground Quarries*

Building a tunnel through ground honeycombed with old quarry workings was one of the difficult and notable pieces of work referred to above. The line which extends from the Bastille to the Porte Saint-Gervais intersects the old underground gypsum quarries beneath the Buttes Chaumont Park. These quarries are great galleries about 33 ft. wide and 33 ft. high, having the roof supported by pillars 22 to 26 ft. diameter.

At the station adjacent to this park there is only one line of galleries, the floor of which is 110 ft. below the surface, while the rail level is at about 100 ft. The concrete tunnel approaching the station has a semicircular roof arch, in order to reduce the lateral thrust, and the arch has a thickness of 3.28 ft. at the crown and 4.26 ft. at the springing. The footings are reinforced, and are supported on concrete piers 5 ft. diameter and 19.7 ft. apart, which extend to the rock bed of the quarry. On the piers are brackets supporting transverse girders which in turn carry the invert of the tunnel (Fig. 1). The open spaces around these tunnel supports were partly filled.

In approaching the twin tunnels of the station proper, the tunnel is widened to a total width of 33 ft. between the faces. The thickness of the arch is 3.6 ft. at the crown and 5.75 ft. at the springing. In view of the heavy load sustained, the sidewalls are carried to the bed of the quarry, and at one point it has been necessary to put in a central pillar or support to reinforce the arch, as shown in Fig. 2. Further, in the open galleries, the heavy walls (lacking any side bearing) were still insufficient to resist the thrust of the heavily loaded arch, and it was necessary to back them with concrete buttresses built in horizontal drifts driven into the loose fill and extending to the gypsum pillars of the quarry. These horizontal buttresses are about 6½ ft. square and from 26 to 50 ft. long. They are shown in Figs. 2 and 3. In the station (Fig. 3), each of the twin tunnels is 24.6 ft. wide (for track and platform), and the dividing wall is pierced with arched openings. The side and center walls are carried down to the quarry floor, and the invert is supported partly by the old quarry roof pillars (where these are intersected by the tunnel) and partly by intermediate columns or piers. The inner portion of the invert is reinforced by steel rods near the lower face. Horizontal buttresses are used here also, to resist the arch thrust. Fig. 4 shows the location of these buttresses along the line of the tunnel.

In following under the Rue du General Brunet, the line is at a higher level, and is above three stages of the quarry galleries, as shown in Fig. 5. Here it was necessary to support the tunnel on foundations carried down 115 ft. to the rock bed of the quarries, or 60 to 85 ft. below the rail level. Two rows of wells were sunk, 31 ft. c. to c. transversely and 20 ft. c. to c. longitudinally (Fig. 6). There are 220 of these pier bents, ranging in height from 100 to 140 ft. Each pier is in two parts, the lower portion being 7 ft. diameter. As these penetrated old filling which produced noxious gases, special ventilating methods were necessary. The upper part of the well, above the level of the tunnel invert, was rectangular, 7x9 ft., with the greater dimension transverse to the tunnel. In the well was placed a form for the inner face of the side wall and the springing of the roof arch. Concrete was then filled into the well to within 6½ ft. of the surface of the ground, thus forming a guard wall, or retaining wall, to prevent movement of the surrounding soil. The method of advancing the tunnel between these rows of concrete piers (forming 8-ft. length of the side walls, 12 ft. apart longitudinally) is shown in Fig. 7. From a shaft on the center line of the tunnel a rectangular heading was driven (section *A-B*), having its top level with the top of the outside of the tunnel arch. This was widened laterally (*C-D*) until the stone masonry arch could be built, fitting the recesses for the springing formed in the concrete side walls previously built in the shafts. The arch was completed for a length of 8 ft., the length of the well. Then the heading was pushed forward for the space of 12 ft. between the wells, and widened laterally (*E-F*). A concrete haunch block was built at each side, to carry the stone arch, the top of this block being some 5 ft. above the springing line (*G-H*). With the roof thus completed, the excavation of the core beneath it was undertaken, and the haunches were underpinned while the side walls were built beneath them in the 12-ft. lengths between adjacent wells. The final step was to build the invert, resting on ledges or brackets on the piers (*I-J*). After the completion of the work, the stone masonry of the roof arch was thoroughly grouted with cement.

In the twin tunnel at the station under the Place du Danube three rows of piers were required (Fig. 8). It will be seen that the entire structure is in effect an underground concrete viaduct.

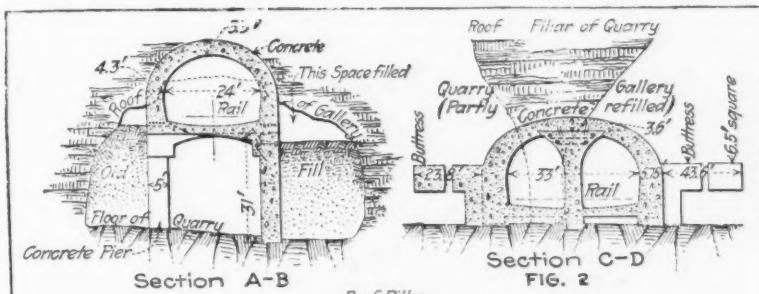
Tunnel of Unsymmetrical Section

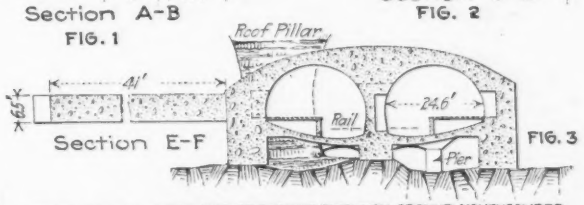
At the Rue Mirabeau is the junction of the Auteuil terminal loop, and in order to avoid endangering the shallow foundations of the old Auteuil Church, it was necessary to bring the single track on this side of the loop as near the surface as possible by a sharp grade up from the junction, while the other side of the loop is at normal grade (Fig. 9). The tunnel at the station, where the two single-track lines at different grades approach to form a double-track line, is of very peculiar section, somewhat resembling an egg laid upon its side. At the smaller side is a platform serving one track, while at the larger side is the inclined roadbed for the second track. The roof and side walls are of stone masonry, while the invert is of brick (four rings, laid on end) on a bed of concrete. The construction of this work in loose and water-bearing soil, and close to important buildings, was a difficult and delicate task.

Tunnel Supporting Bridge Piers

The Ceinture Ry. (or Belt Line) of Paris crosses the Rue d'Auteuil by a bridge having four girders carried on abutments 100 ft. apart and two intermediate rows

*See "Engineering News," Sept. 21, 1911, p. 336, for previous account of this work.





FIGS. 1, 2, 3, SECTIONS OF TUNNEL THROUGH GROUND HONEYCOMBED WITH OLD QUARRY GALLERIES; METROPOLITAN RAILWAY, PARIS.

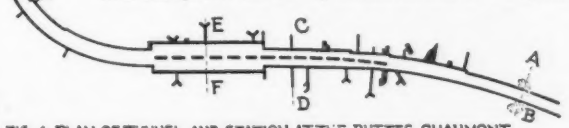


FIG. 4 PLAN OF TUNNEL AND STATION AT THE BUTTES CHAUMONT, SHOWING HORIZONTAL BUTTRESSES TO BRACE THE TUNNEL ARCH IN CROSSING OLD UNDERGROUND GALLERIES

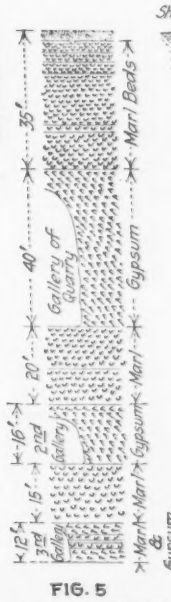


FIG. 5

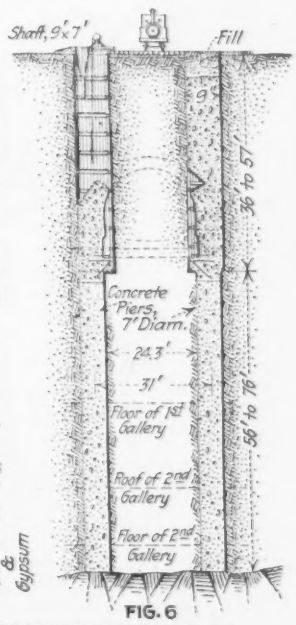


FIG. 6

FIG. 5 SECTION OF GROUND TRAVERSED BY THE METROPOLITAN RY. TUNNEL AT THE RUE DU GENERAL BRUNET, PARIS (SHOWING THREE LEVELS OF QUARRY GALLERIES BELOW THE TUNNEL). FIG. 6 METHOD OF SUPPORTING TUNNEL ON CONCRETE PIERS

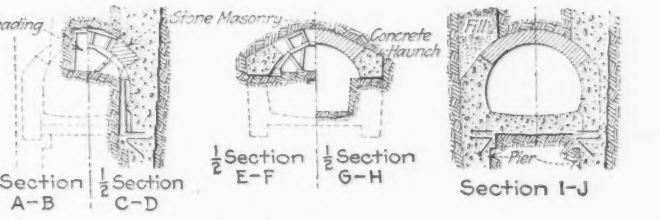


FIG. 7 METHOD OF DRIVING TUNNEL UNDER THE RUE DU GENERAL BRUNET, AT PARIS

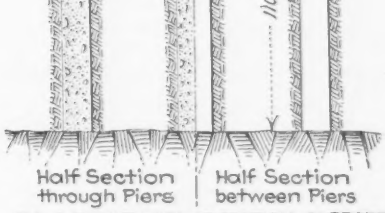
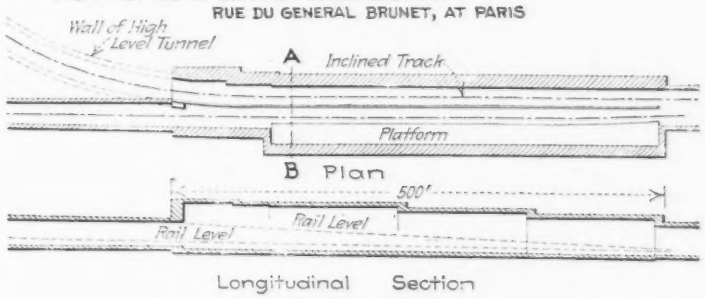


FIG. 8 TWIN TUNNEL AT STATION, SUPPORTED ON THREE LINES OF CONCRETE PIERS WITH CROSS GIRDERS



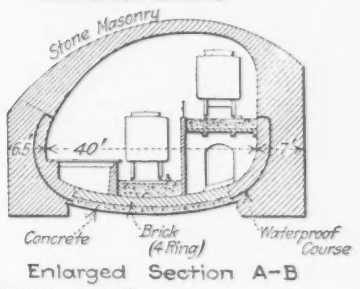


FIG. 9 UNSYMMETRICAL TUNNEL SECTION WITH TRACKS AT DIFFERENT LEVELS

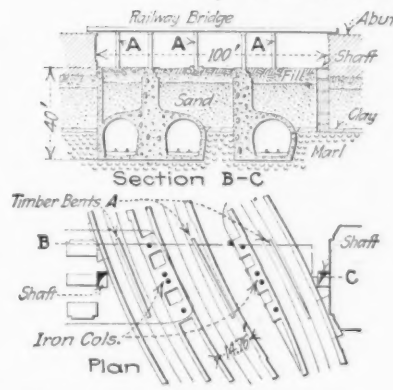


FIG. 10 TUNNEL WALLS SUPPORTING COLUMNS OF RAILWAY BRIDGE

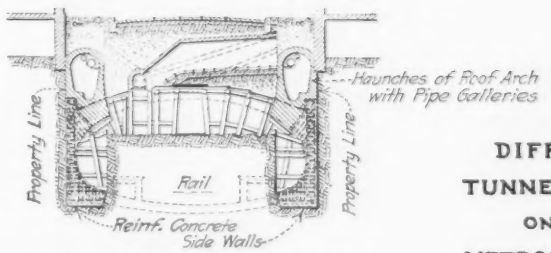


FIG. 11 WIDE TUNNEL (WITH THIN WALLS OF REINFORCED CONCRETE) BETWEEN PROPERTY LINES IN A NARROW STREET; METROPOLITAN RAILWAY, PARIS.

DIFFICULT TUNNEL WORK ON THE METROPOLITAN RAILWAY, PARIS. (FRANCE)

ENG. NEWS

of cast-iron columns. The Metropolitan Ry. in passing under this bridge was located with a view to supporting these columns on the tunnel walls, in order to avoid any alterations of the bridge, which would have been very costly. As shown in Fig. 10 there are three tracks, with one twin tunnel and one single tunnel. The center wall of the former and one of the side walls of the latter form of foundation piers for the bridge columns.

These foundations, however, were built before the tunnels were driven, and have their bases 10 ft. below the ground. During this work, the columns of each row

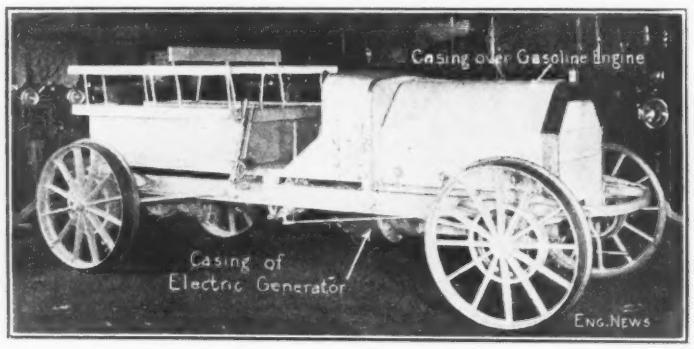


FIG. 1. GASOLINE RAILWAY MOTOR TRUCK WITH ELECTRIC GENERATOR FOR OPERATING TOOLS FOR BORING TIES AND DRIVING SCREW SPIKES

(T. W. Snow Construction Co., Chicago: Builder.)

were removed in pairs, the bridge girders being supported temporarily by three timber bents *A*, as shown.

SEGMENTAL TUNNEL WITH THIN SIDE WALLS

The station at the Rue de la Crimée is located under a narrow street, with a width of only 46 ft. between building foundations. To avoid interference with the building and to provide the 41.6 ft. of width necessary for a double-track station, with platforms, left only 26 in. for side walls. The construction adopted was to use a vertical wall of this thickness, enlarged on the upper and lower edges to form the springing of the roof and invert arches, this wall being of concrete heavily reinforced. Above the haunches of the roof arch (and formed with them) are pipe galleries serving the property affected. The arch is 28 in. thick at the crown, with a depth of cover of 12 ft. This construction is shown in Fig. 11.

The side wall, haunch and gallery were built first along one side of the work, in lengths of 10½ ft. (not continuous), in open shafts or pits. The other side of the work was then treated in the same way. Then a heading was driven at the level of the upper part of the roof arch, for the entire length of the work. This was widened in lengths of 10 ft. until it extended between the haunches previously built, and the arch then built in these lengths. The roof was thus built in 10-ft. lengths, care being taken not to work on any one length until the arch in the adjacent length had thoroughly set. After this, the core or dumpling was excavated and the invert built between the bottom haunches of the side walls.

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A Standard Specification for Stucco entitled "Standard Specifications for Exterior Plastering (Stucco)" has been issued, under date of August, 1913, by the Associated Metal Lath Manufacturers. Copies may be obtained by writing the association at 812 Wick Building, Youngstown, Ohio.

An Electric Tie Boring and Spiking Machine

While the use of the screw spike is increasing but slowly in this country, in spite of the advantages of this form of rail fastening, it is used to a sufficient extent on some roads to warrant the introduction of special appliances for boring the ties and putting in the spikes. The work may be done by hand, by the use of hand augers and crank-handle socket wrenches, but this is

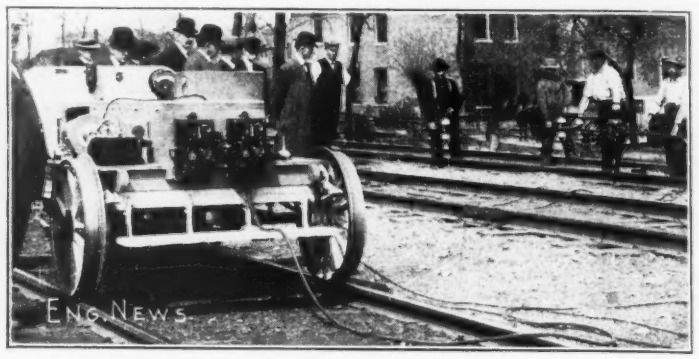


FIG. 2. REAR VIEW OF MOTOR CAR, SHOWING CABLE CONNECTIONS TO TRACK TOOLS

slow and laborious, and if the work is more than experimental it can be done to better advantage by the use of mechanical appliances. A gasoline-motor car operating tools for track work by means of flexible shafting was described in our issue of April 2, 1908, and other similar machines were described and illustrated in our issue of Oct. 19, 1911.

Augers and wrenches operated by compressed air and electric motors are in extensive use for a great variety of purposes, and on street-railway track current for these tools can be taken direct from the trolley wires, as described in ENGINEERING NEWS of Nov. 3, 1910, in connection with the reconstruction work on the Chicago lines. To adapt the tools to ordinary railway work the only special equipment required is a car carrying an air compressor or electric generator, together with the necessary motive power and propelling gear.

A machine of this class for operating electric tools is shown in Fig. 1. It is a gasoline motor-car equipped with an electric generator, and mounted on four 30-in. wheels, the wheels being larger than those used for railway section motor cars. The car is equipped with a four-cylinder 40-hp. gasoline engine, with automobile control, and runs on the rails under its own power. When engaged in track work, the engine drives a generator of 7½ kw., mounted in the rear of the engine hood, and just in front of the floor of the driver's seat. This furnishes current for five motor tools; three for boring holes in the ties and two for screwing in the spikes. A cable enables the tools to be worked at a distance of 1000 ft. from the car, and a cable reel is mounted on the side of the car, as may be seen in Fig. 2, which shows the rear of the car and men holding three tools. Behind the driver's seat is a body with two longitudinal seats, holding 10 men. The weight of the car complete is 3400 lb., and a simple turntable is provided so that the car may be easily removed from the track.

This machine has been in use for nearly two years on

the Chicago, Rock Island & Pacific Ry. We are informed by officers of this road that the machine has given very satisfactory results and will average boring and driving from 2000 to 4000 spikes per day. The work done depends upon the number of men employed, the number of drilling and boring machines, and the amount of additional work that the men do besides what is strictly chargeable to boring and driving. With three men drilling and two men driving and doing nothing else, the machine will put down 3000 spikes per day. Ordinarily, however, they cannot work continually and they will put down anywhere from 2000 to 2500 spikes per day. The men are obliged to distribute the tie-plates and screw spikes in addition to boring the holes. The machine was built by the T. W. Snow Construction Co., of Chicago.

A Rotary Air Pump for High-Vacuum Condenser Service*

At various times in the past, different schemes have been noted in ENGINEERING NEWS for using simple and less expensive apparatus effectively to remove air and vapors from condensing systems of steam turbines and to maintain that high degree of vacuum which has been found advantageous.

One more such device, based on designs of the Allgemeine Electricitäts Gesellshaft of Berlin, Germany, has been put out by the Wheeler Condenser & Engineering Co., of Carteret, N. J. This is shown by the accompanying figures. There is first a small high-speed impeller with a number of ports on its periphery and this is rotated in the opposite direction from that of an ordinary centrifugal-pump impeller. Outside this is a ring with channels of decreasing cross-sectional area and outside of the ring, finally, is a volute discharge casing. The impeller and compression ring are shown diagrammatically in Fig. 1. Air is admitted around the entire periphery of the impeller and is drawn into the tapered channels of the ring partly by the aspirating effect of the streams from the impeller and partly by direct trapping. The shape and dimensions of the compressor ring and the proportional relations between the impeller and the ring are such that the streams from the impeller are broken up into slugs in the compression channels. On account of the decreasing cross-section of the latter, the entrapped air is compressed by the moving water and the resulting mixture is finally discharged through the volute casing to a large outlet pipe.

These air pumps are being made for both jet and surface condenser systems. For the latter, the air and condenser pumps are being built in combined units though all of the various parts of each are still distinct as shown in Fig. 2. Air and condensate are drawn from the condenser through the same pipe; just inside the casing connection, the air is drawn off above the condensate and led down to the vacuum-pump rotor while the condensate flows along and down to the impeller of the condensate pump. This water is forced through a connection on the side of the casing chamber, while the mixture of air and forcing water from the vacuum pump is discharged downward from its casing chamber. So far as their functions go the pumps are independent. Economy results from saving in cost of piping, in power for driving, and in space demanded by auxiliaries.

On account of the positive aspirating effect of such devices the vacuum is not entirely lost when there is excessive air leakage, etc., but it is reduced according to the maximum amount which the pump can aspirate.

It is claimed that a vacuum can be secured equivalent to about 98% of the theoretical height of mercury column for temperatures of the aspirating water, over a temperature range of the latter from 40° to 90° F. In Fig. 3 are shown plotted data from a manufacturer's test of one of these pumps. The aspirating water was at 78° F. for which the theoretical absolute pressure was about 0.96 in. of mercury compared with 1.1 in. secured by the pump or about 0.5% the height of the vacuum-gage mercury column. Nothing is said about the power required to drive the pump under these tests.

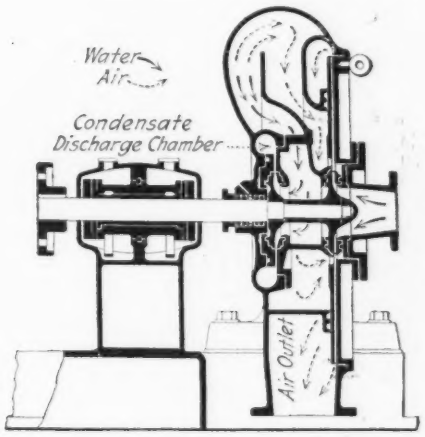


FIG. 1. DIAGRAMMATIC ARRANGEMENT OF IMPELLER AND COMPRESSION RING FOR ROTARY AIR-PUMP

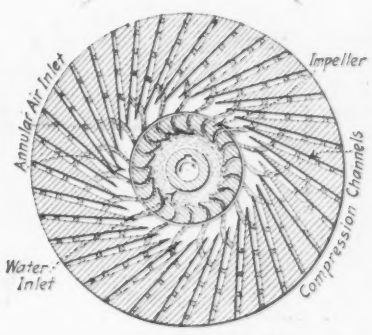


FIG. 2. CROSS-SECTION THROUGH COMBINED AIR AND CONDENSATE PUMP

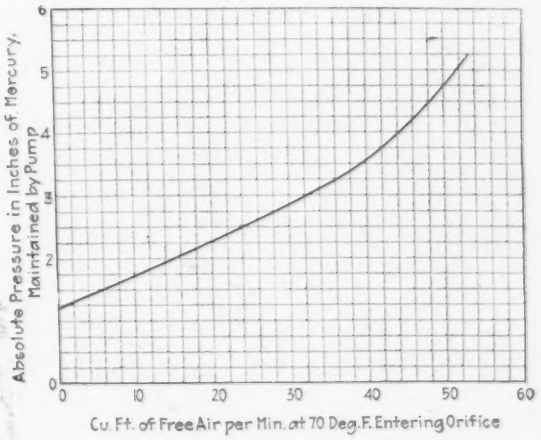


FIG. 3. PERFORMANCE CURVE OF WHEELER ROTARY AIR PUMP

*From information furnished by the Wheeler Condenser & Engineering Co., Cartaret, N. J.

A Bridge Specification for highway bridges, intended to aid "local highway officials in fixing requirements to which bridges must conform" has been issued by the U. S. office of Public Roads. It may be obtained by writing to the Superintendent of Documents, Government Printing Office, Washington, D. C., for Circular No. 100, office of Public Roads, entitled "Typical Specifications for the Fabrication and Erection of Steel Highway Bridges."

Rapid Construction on Medina Valley Irrigation Project in Texas

By Terrell Bartlett*

A concrete dam of large size has recently been completed near San Antonio, Tex., in the remarkably short time of one year. The dam is 164 ft. high and contains nearly 300,000 cu.yd. The first concrete was placed on Nov. 9, 1911, and the concreting completed, plant removed and site cleared up by Dec. 31, 1912. The dam is a part of the Medina Valley Irrigation Project, of which the location and general features are shown in the accompanying map (Fig. 2).

This project is based upon the storage of the flood waters of the upper Medina River by means of the main dam, which is located at the head of the cañon through which the stream emerges from a region of rugged limestone hills onto the rolling coastal plains of southeastern Texas. The lands to be irrigated lie in a solid body of 60,000 acres in the plain country at a distance of from 20 to 30 miles from the main dam. The lands are of remarkable fertility, are well drained, and have exceptional transportation facilities, lying along and between two trunk-line railways, the Southern Pacific and the International & Great Northern. The water is conducted from the main dam to the lands first in the natural river bed for the four miles of cañon, which will be converted into a narrow lake by means of a secondary or diversion dam. This structure is 50 ft. high and turns the water into the canal head. From this point the main canal extends 24 miles to the head of the distributing system.

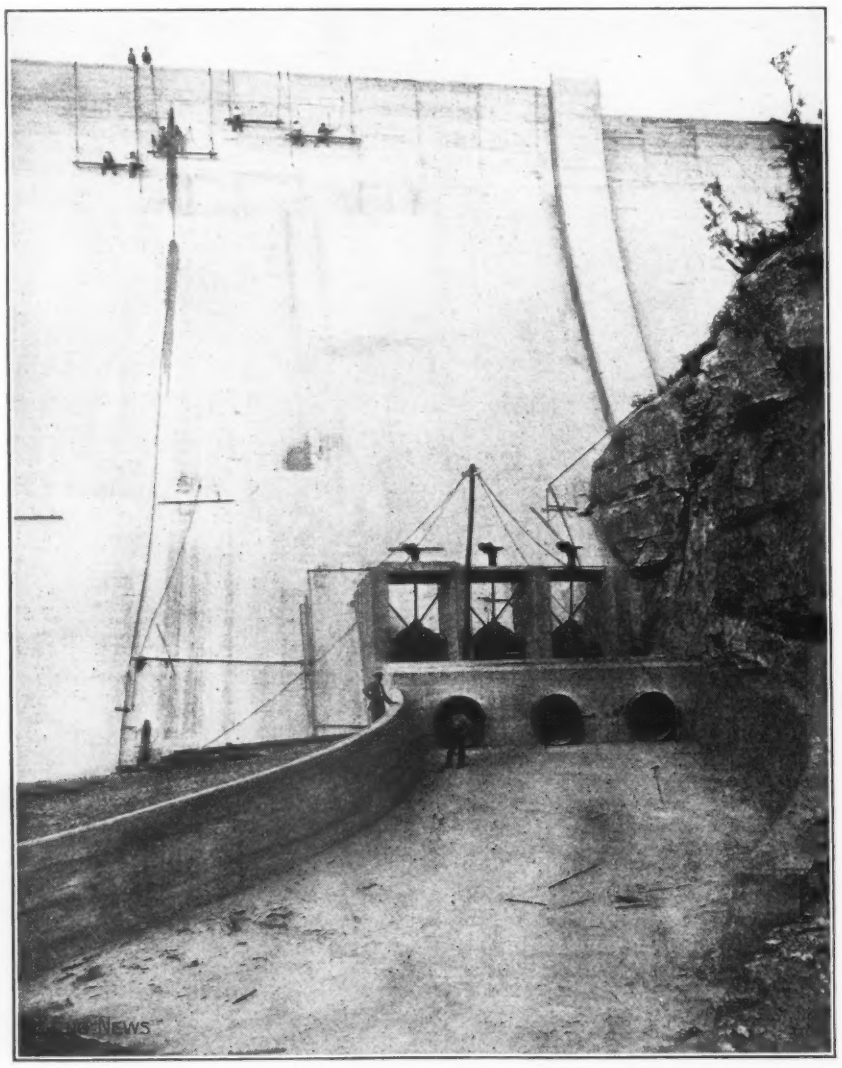


Fig. 1. The 5-Ft. Main Outlet Gates of the Main Dam of the Medina Valley Irrigation Co.

Construction of Main Dam

The cubical contents of the main dam is nearly 90% of that of the famous Roosevelt Dam near Phœnix, Ariz. The Medina Dam is 164 ft. high above the river-bed, and from the bottom of the cutoff trench to the top of the concrete has a total height of 180 ft. The crest length of the dam is 1580 ft., the base width is 128 ft., and the crest width 25 ft. The plan and cross-section of the dam, Fig. 3, show in detail the dimensions and Figs. 6 to 9 are views of the dam at various stages of its construction. The foundation is in the massive level-bedded limestones, which form the cañon walls and all the hills of the region. The spillway, 1200 ft. long, is over a saddle into an adjoining ravine.

The dam-site is about 14 miles by airline from Dunlay, the nearest point on the Southern Pacific Ry. and its nearest wagon-haul was originally from La Coste, a distance of 23 miles. When the bonds for the project were sold in May, 1911, excavation for the foundation was immediately commenced, and material for camp buildings, machinery foundations, etc., was hauled in by a new wagon road from Dunlay. At the same time construction was rushed on a standard-gage railway from Dunlay to the dam. On account of the rough character of the country this line is 19½ miles long, with sharp curves, and grades of 3% against the traffic and 3½% against the returning empties. While this railway was in process of construction, the excavation and lighter preliminary construction, as well as the opening of quarries, was going forward at the dam-site, and simultaneously all the machinery was assembled and heavy timbering was framed at Dunlay.

The railroad was completed the latter part of August, and the plant was erected within sixty days thereafter, a really remarkable performance when the unusual magnitude of the plant and its isolation are considered.

The materials for making the concrete were all secured at the site with the exception of the cement. The quarries are in a very hard, cherty, semi-crystalline limestone. Quartz sand of a mediocre quality could have been delivered over the railway for about $1.40 per cu.yd., but it would have been very difficult to handle the large quantity of sand over the single track, steep-grade road. The fines and dust from the first crushing of the limestone together with dust from special pulverizing crushers was utilized for sand throughout the dam. The cement used

*Of Bartlett & Ranney, Consulting Engineers, Gibbs Building, San Antonio, Tex.

came principally from the mills of three cement companies, located in the State of Texas, making as short a haul as possible.

The mixture of concrete used was 1 cement to 3½ sand (limestone dust and fines) to 6½ stone. This required slightly over 1 bbl. of cement per cu.yd. Daily tests of cubes and of short, plain beams showed a strength of great uniformity, greater than many first-class concretes of 1:3:6 mix. After the dam had reached a height above the zones of heaviest pressures it was, therefore, deemed justifiable to decrease the percentage of cement.

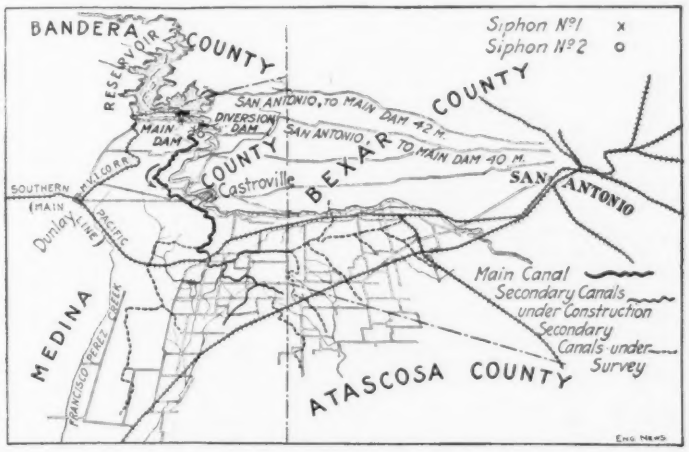


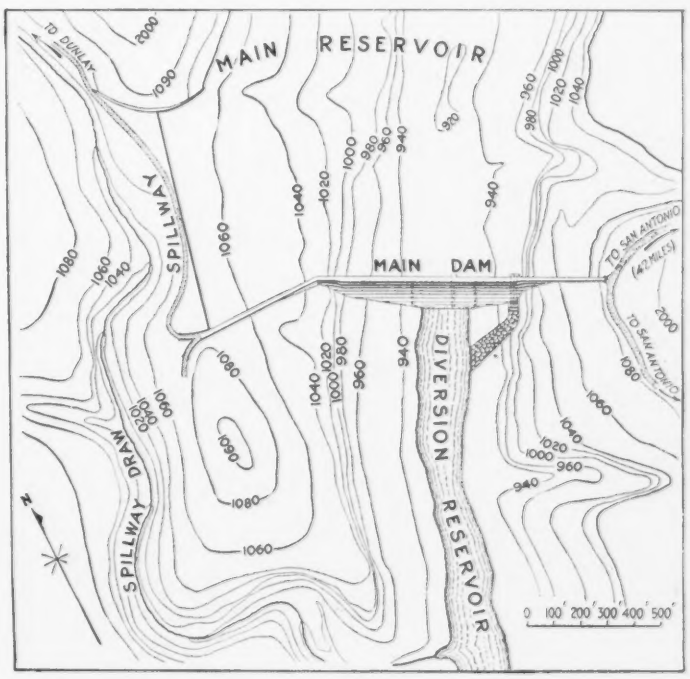
FIG. 2. SKETCH MAP OF THE MEDINA VALLEY IRRIGATION CO. SYSTEM

Test specimens with a smaller proportion of cement showed a much greater strength than usual in such mass construction as well as a high degree of imperviousness. When this decrease was tried in the dam it was found to be detrimental to the plasticity of the concrete and its ease in working, and on this account the attempted economy was abandoned and the original proportion used for the entire dam. Rubble stones up to ten tons weight were imbedded in the concrete. At the bottom from 10 to 15% of the mass consisted of these "plums," but toward the top the percentage gradually decreased. The percentage for the entire dam was 9.9%. A larger percentage of plums would have effected a cost saving in the items of cement, crushing and mixing, but would have delayed the rapid progress, and thus overbalanced the saving by the loss in overhead expense and interest charges.

CONTRACTOR'S PLANT—The general layout of the plant is shown by Fig. 4. The location on the west bluff was dictated by the railway location, which necessarily came in on that side. Adjacent to the end of the railway on one side were the storerooms and the commissary and opposite were the shops and lumber yards. A spur track on a trestle was carried out over the boiler plant and over the center of the cement shed, enabling unloading of fuel oil and cement by gravity. Power was derived from seven 150-hp. boilers, with steam lines to the various engine locations. Only the small engine for lighting and the compressors were located at the steam plant.

The workings of the plant will be best understood in connection with the layout plan (Fig. 4) by briefly tracing the movement of material from the quarry to the dam. The two principal quarries were located above the dam on the plant side, the furthest at a distance of 2000 ft. A third quarry exclusively for plums was located in the east bluff below the dam. Skips of stone and plums from the main quarries were handled by three standard-gage locomotive cranes onto 4-yd. dump cars. The latter were hauled around a loop track to the crushers by four dinkies.

The crushers were two 7½ D Gates gyratory located immediately back of the combined mixer bin and screen house. Bucket conveyors took the crusher run stone to the screens above the stone and sand bins (Fig. 5). The desired percentage of medium-sized stone was returned to two disk crushers and two hammer pulverizers for making the sand. Under the bins was a mixing deck with five hoppers served by radial cutoff gates from the stone bins, and automatic measuring chutes for sand. Cement was delivered to this deck by a belt conveyor from the adjacent shed. On the next deck below was a battery of five 1-yd. batch mixers emptying into hoppers. The crushers, conveyors, sand machines,



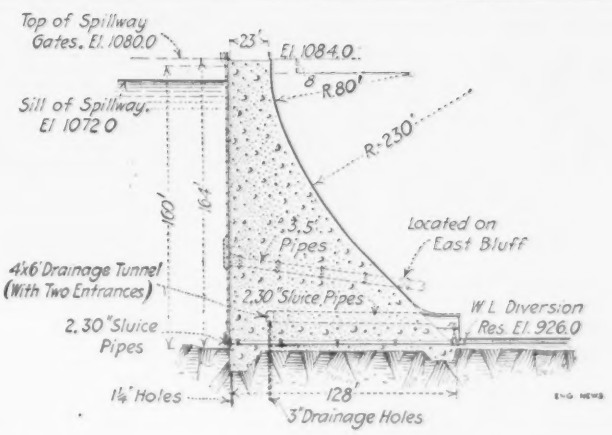


FIG. 3. PLAN AND CROSS-SECTION OF THE MAIN DAM, MEDINA VALLEY IRRIGATION CO.

shafting and driving engines for this machinery, as well as for the mixers, was a complete double installation so that no delays were suffered from breakdowns and replacement of worn parts. On still a third level below was a narrow-gage track on which flat cars carried the concrete buckets about 75 ft. to the cableways. This method of delivery was used exclusively for the narrow upper portion of the dam. For the principal mass of the

dam delivery was almost entirely by inclined chutes leading from the discharge hoppers down the face of the bluff into a large wooden hopper located over the end of a double-track incline, which extended from bluff to bluff, first along the downstream edge of the dam, and later at a high level on the face of the dam. (See Figs. 6 and 8.) The two cableways and control were of the Lidgerwood type, cables 2¼ in. with spans of 1050 and 1270 ft. Their load capacity was ten tons.

A car carrying two 2-yd. buckets or 4 yd. per trip was used on each incline track, so that it was possible to send 8 cu.yd. at once to the various derricks by the incline, as well as 2 yd. simultaneously by each of the cables. The phenomenal progress records were made possible, first, by the adequacy of the crushing and mixing plant, second, by the triple system of hoppers by means of which the materials were always ready and waiting for the buckets, and third by the incline with its enormous and rapid delivery capacity. The limiting speed condition was in the quarry output. The plant was in such excellent condition on completing the work that the entire outfit has been taken to Spain for use on the work of the Ebro Irrigation & Power Co., which is constructing several large dams near Barcelona.

Records—The following records were made: In ten hours 2240 cu.yd. of rubble masonry (about 10% plums). The best monthly record, with one 10-hour shift, and no Sunday work, was 40,303 yd., and for the three months of June, July and August, 1912, the average yardage was 35,850.

Other Structures

The Diversion Dam (Figs. 10 and 11), also of rubble concrete, is an overflow wier with ogee crest and roll-way. It is 50 ft. high, 44 ft. wide at the base, 440 ft. long, and is arched slightly upstream. The construction plant was similar in general scheme to that of the main dam, except on a much smaller scale, as the diversion dam contains only about one-tenth of the mass of the former structure. An interesting point of difference was in the utilization of fixed guy derricks instead of movable stiff-legs. These derricks were placed on pedestals of concrete within easy reach of each other and it was possible to so place the guys as to avoid interference with booms or cableway (Fig. 11). The lower portion of the pedestals became part of the structure and the upper part, originally made hollow by a core of empty barrels, was removed upon completion of the work.

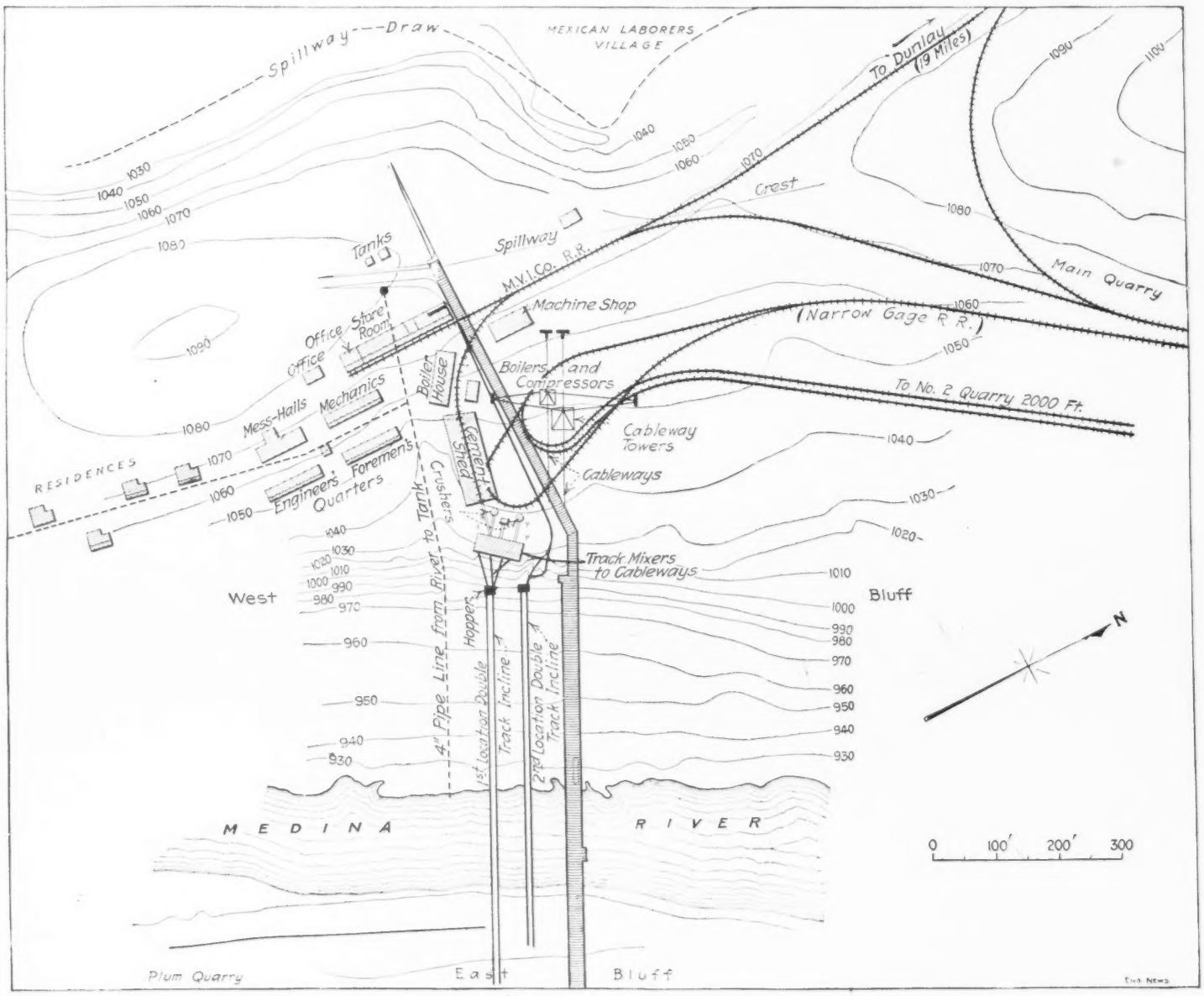


Fig. 4. Layout of Construction Plant, Medina Dam

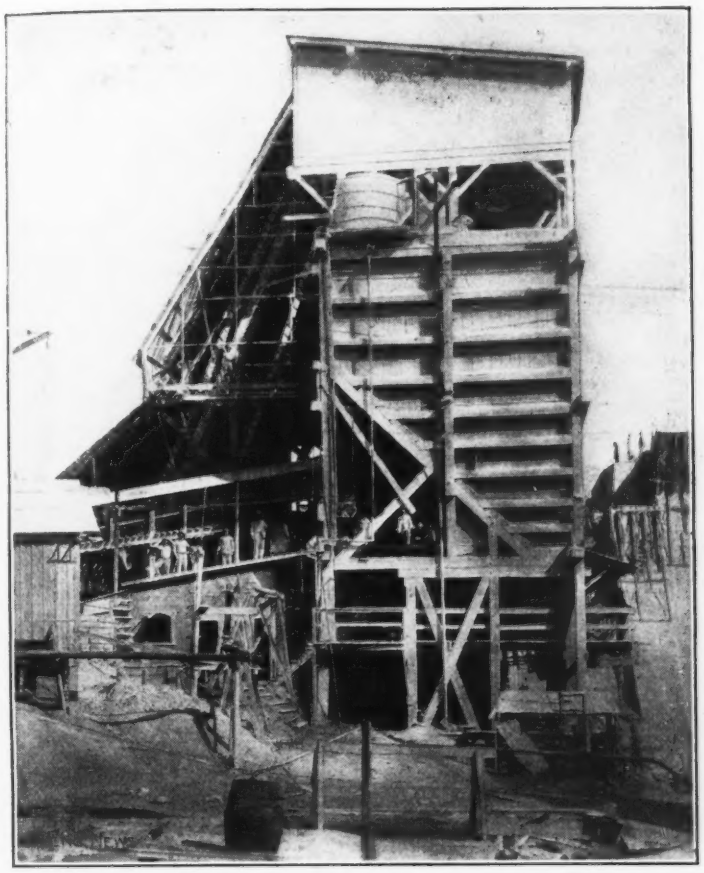


Fig. 5. Crushing, Screening and Mixing Plant, Medina Dam

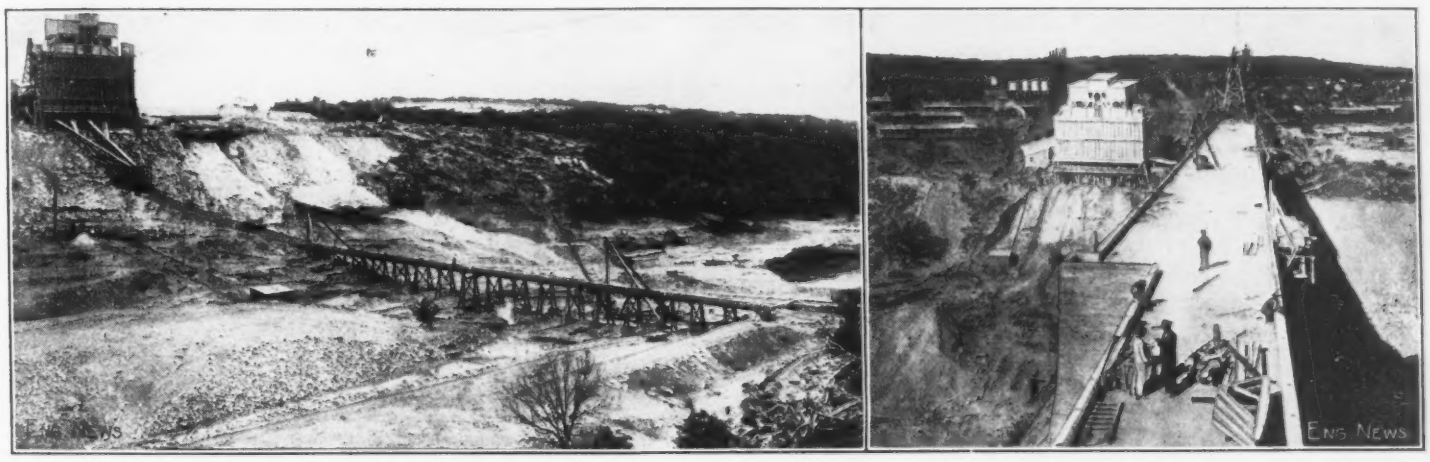
The Headworks with a double set of metallic gates is shown by the view Fig. 12.

The Canal is of ordinary construction with about 3000 ft. of concrete-lined channel in rock cut, the remainder being almost entirely sidehill work in a compact clay. The canal capacity is 600 sec.-ft. The siphons and flumes along the main canal line are worthy of note.

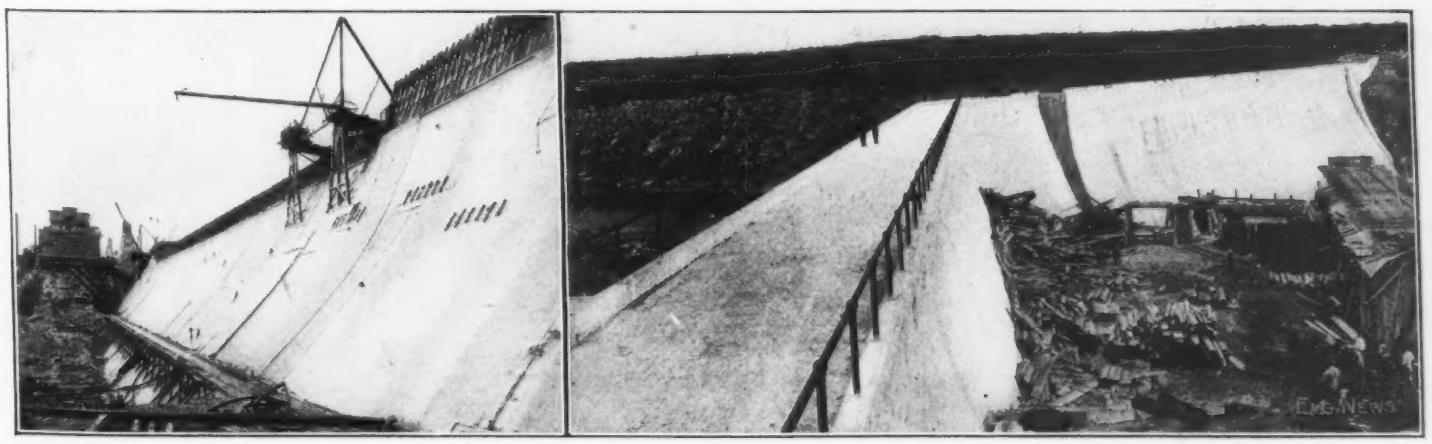
Reinforced-Concrete Siphons—The headworks and first mile and a half of the canal are on the west bank of the river. The canal is then carried under the river to the east bank in a reinforced-concrete siphon consisting of two bores 8 ft. in diameter (Fig. 13). After another mile the canal is returned to the west bank by means of a similar siphon. Two miles of extremely rough country is thus avoided, and the distance shortened about a mile.

Flumes—There are in all eleven flumes along the main canal, aggregating slightly over one mile of flume length. These flumes consist of creosoted timber trestle of long-leaf Texas pine with truss and tower construction, where high, supporting two No. 180 semicircular Hess flumes 9½ ft. in diameter, of galvanized toneau metal. The foundations are on concrete pedestals. All timber was framed and bored before receiving the 12-lb. creosote treatment. The largest flume, 95 ft. high and 1500 ft. long, is shown by Fig. 14.

General—The distribution system is simple and involves no unusual features except the Chacon Dam, an earth structure which will collect return seepage waters



Figs. 6 and 7. The Medina Dam Site on Nov. 6, 1911, and the Nearly Completed Dam, Nov. 13, 1912



Figs. 8 and 9. Downstream Face of Medina Dam, Sept. 12, 1912, and General View of Dam, December, 1912

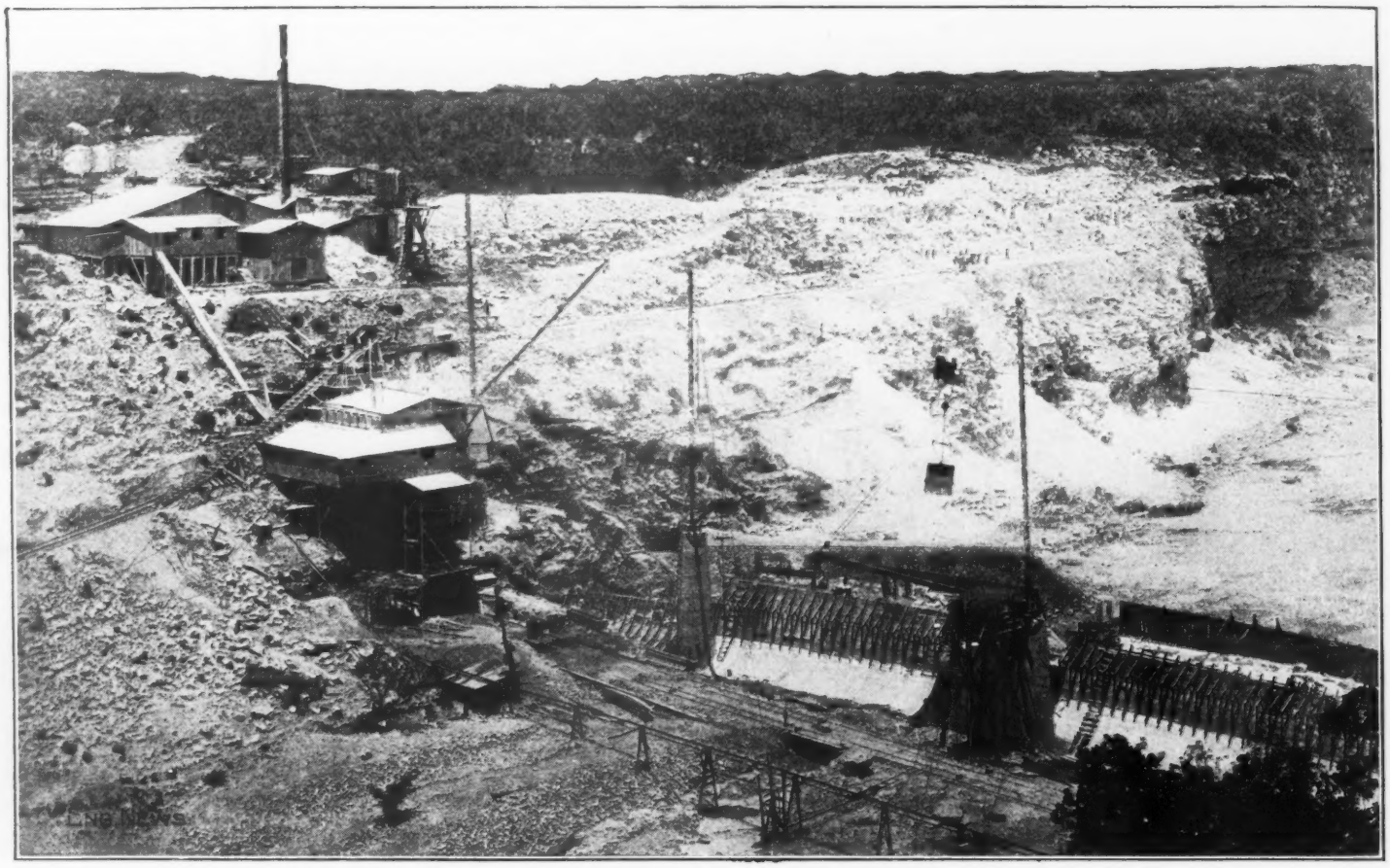


FIG. 10. MEDINA DIVERSION DAM UNDER CONSTRUCTION, MAY 28, 1912; LOOKING UPSTREAM

from several thousand acres and storm rainfall from about 70 sq.mi. and will also form an equalizing reservoir on one of the principal laterals. No elaborate system of water measurement will be installed until the farms have attained to the very intensive development necessary to justify the considerable cost of automatic and recording water meters.

An unusual feature of the project is the enormous storage of the main reservoir, which has a maximum capacity of 300,000 acre-feet. This large storage was considered advisable on account of the torrential nature of the stream, and it will make possible the conservation of a large percentage of the runoff from the Medina Basin.

PERSONNEL

The credit for the undertaking is due to Dr. F. S. Pearson, who financed the project and who has directed the entire construction and development work through the Pearson Engineering Corporation of New York. The history of the project is very brief, as the first surveys, investigations and preliminary report were made by Bartlett & Ranney in conjunction with A. Y. Walton, Jr., of San Antonio, as late as the summer of 1910. Special reports on the project were made for Dr. Pearson, by Messrs. John R. Freeman, Thomas Berry, C. S. Slichter, W. O. Crosby, Chester W. Smith and others. The dams were constructed under direct administration by the company and the canal by contract. The

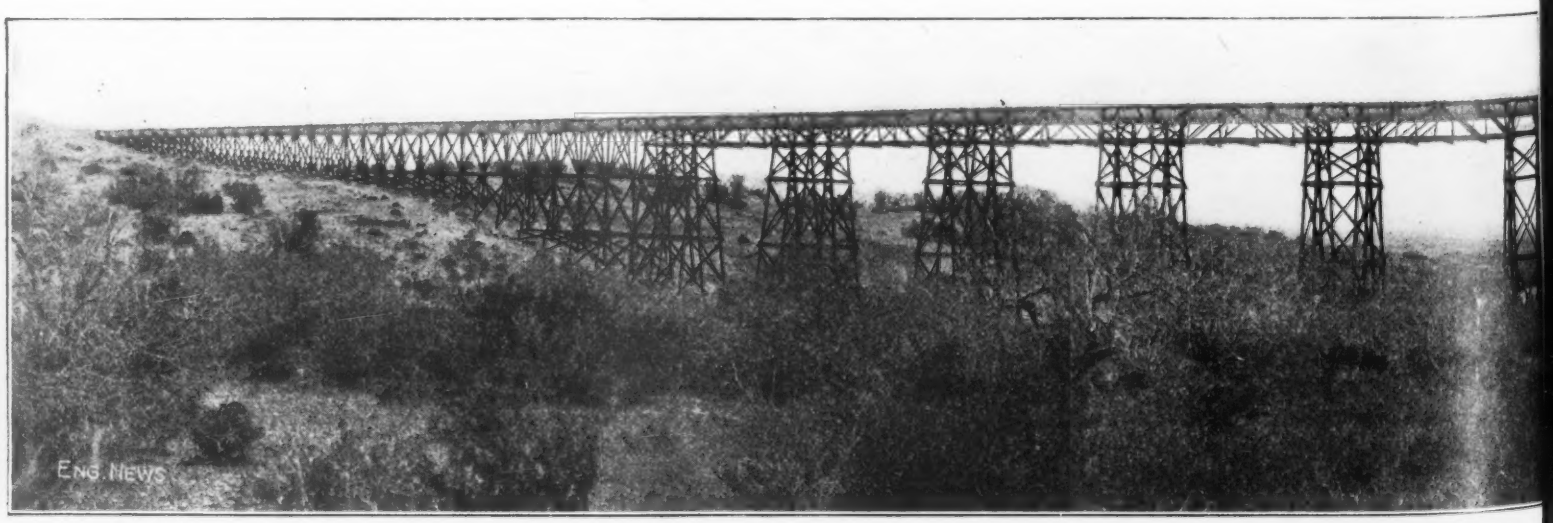


FIG. 14. HESS SEMICIRCULAR METAL FLUME, LONG, MEDINA VALLEY IRRIGATION

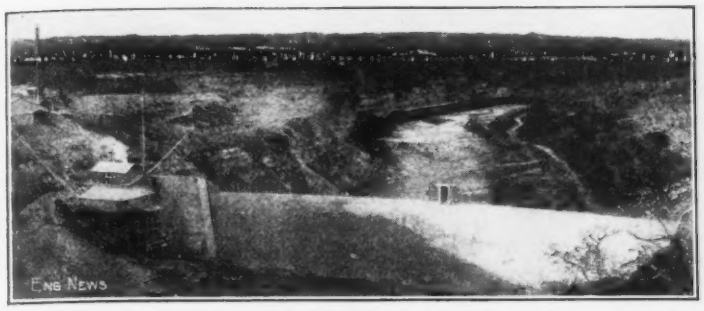


FIG. 11. THE COMPLETED STRUCTURE, NOVEMBER, 1912

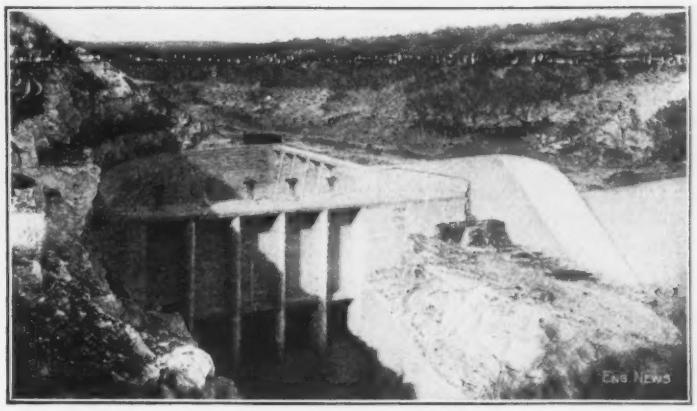


FIG. 12. THE HEADWORKS

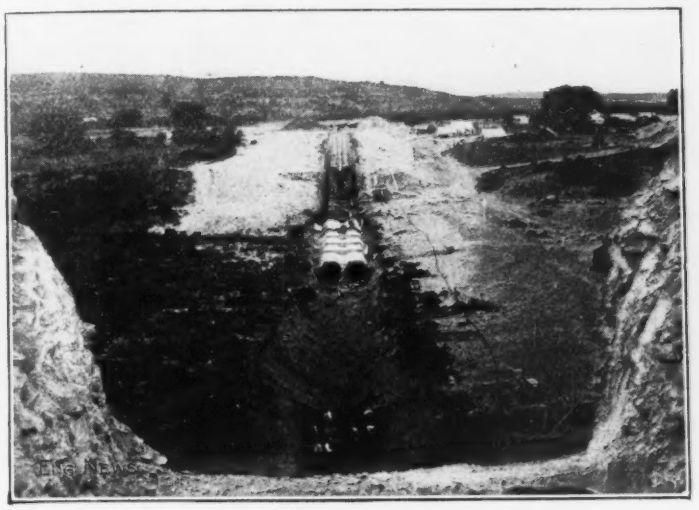


FIG. 13. LOWER SIPHON OF REINFORCED CONCRETE, MEDINA VALLEY IRRIGATION CO.; LOOKING UPSTREAM

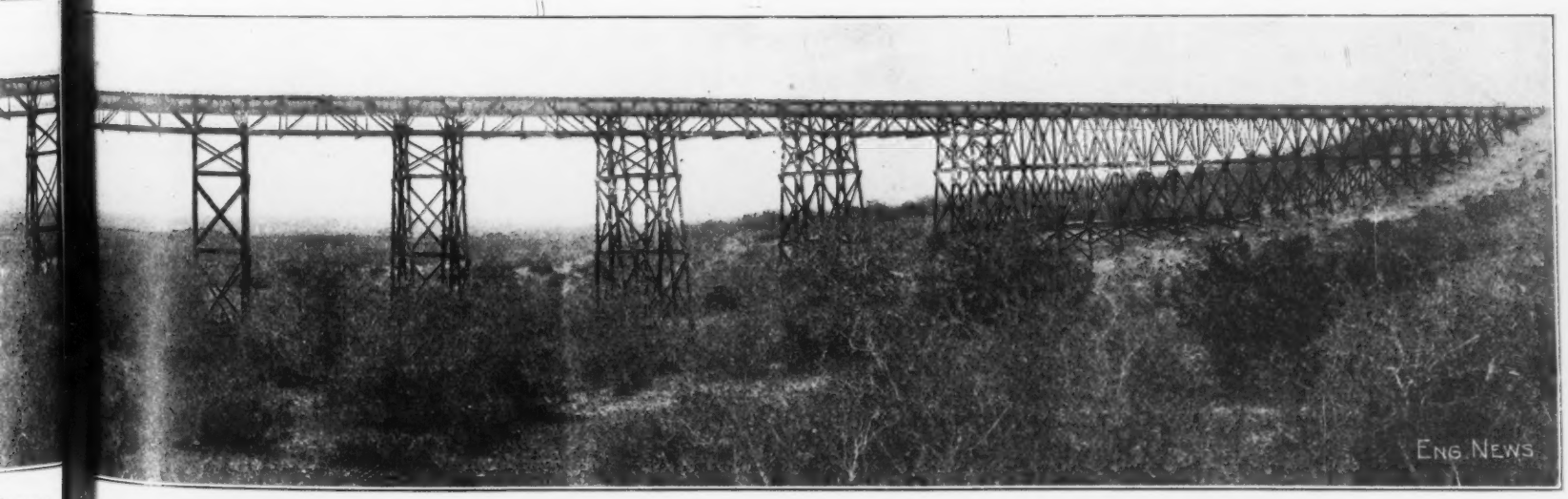
work was done under the general direction of C. H. Kearny and A. W. K. Billings. T. B. Palfrey has been manager of development work, G. W. Caldwell, superintendent of construction, and Willis Ranney resident engineer for the two dams. The canal work has been under J. A. Sargent and B. E. Forbes.

❖

The British Imperial Wireless Telegraph System agreement with the Marconi company (noted as proposed in "Engineering News," Aug. 22, 1912) was accepted by the House of Commons on Aug. 8. The agreement has been under fire and investigation since proposed from those who believed that it was not wise to confine the operation of the government stations to a given system and discouraging as the agreement does, for a term of years, the use of other systems which may develop to be of equal use in long-distance transmission of communications.

In the amended agreement, the company will receive for each station the standard price of $300,000, before fixed, plus or minus a correction for change in price of materials. Interest (2%) on an unpaid third of the purchase money will be allowed during construction. In return for such concessions, the Postmaster-General has the right to use the stations for experimental working. He may cancel the agreement as applying to stations in India, South Africa and Singapore, with a view to introducing some other system though, if a continous-wave system is contemplated, no preference is to be shown another company if the Marconi company can provide such a system of equal efficiency. The royalty payments will be made in respect to each station instead of the system as a whole so that the payments will be affected by patents discarded at any station; moreover, the 10% royalty has been split up and allocated to actual use of Marconi apparatus patents (internal transmitting 4%, external transmitting 2%, internal receiving 2%, external receiving 2%.) so that discard of some parts for another system now affects the revenue.

The clause providing that the Postmaster General must seek the advice of the company before installing a different system, has been struck out. The right of the company to inspect apparatus of any other system where infringement was suspected has been changed to the right to have some independent engineer make the examination. The clause giving the company exclusive privilege of erecting all government long-distance stations for five years has been dropped. A time limit is set on construction and a penalty of $750 per month is fixed for delay in completion. The speed of automatic simplex working has been raised from 50 to 75 words per minute with provision for temporary atmospheric disturbances which render automatic working impracticable. A clause has been added providing that a non-Marconi patent in use at any station may not be acquired by the Marconi company for the prolonging of the royalty.



9½- FT. DIAMETER, 95 FT. HIGH AND 1500 FT.
CO., NEAR SAN ANTONIO, TEX.

A 75-Year Record of Rainfall; St. Paul, Minn.*

The accompanying table and diagrams give the records of a remarkably long, unbroken series of rainfall observations at St. Paul, Minn. The general lack of long-term records in the upper Mississippi region suggests that the ones here given may be of considerable value to a large number of engineers.

Many interesting features are brought out in the diagrams. The greatest recorded annual rainfall (Fig. 1) was in 1849, the amount being 49.69 in. This year also holds the record rainfall for the month of March (Fig. 2) and was the year of one of the greatest Mississippi River floods. The least annual rainfall occurred in 1910, and amounted to only 10.21 in. This gives a maximum difference of 39.48 in., or much more than the mean annual rainfall for the entire period.

The mean annual rainfall for the first 10 years recorded, 1837-1846, is 24.98 in., or 91% of the 75-year mean; the mean of the first 20 years is 25.64 in., 93½% of the 75-year mean; the mean of the first 25 years is 26.46 in., or 96¾% of the 75-year mean; the mean of the first 50 years is 27.33 in., or 99½% of the 75-year mean.

This shows that the mean annual rainfall computed at the end of any of these periods would not have been radically in error from the 75-year mean, in spite of the apparent irregularity in the diagram. The maximum annual rainfall, in 1849, was 181% of the 75-year mean; and the minimum annual rainfall, in 1910, was 37.4% of the 75-year mean.

The two very dry years, 1863 and 1864, caused droughts, according to the United States Weather Bureau, Bulletin Q, which were confined to eastern Iowa, southern Minnesota, southern Wisconsin and portions of Missouri and Kansas. While they were more or less local summer droughts, yet they caused the lowest water hitherto recorded in the Mississippi River from Dubuque to Burlington.

Fig. 2 shows the monthly variation in precipitation, which exhibits even greater contrasts than the compari-

*Tables from the 1912 Report of the Board of Water Commissioners, St. Paul, Minn.

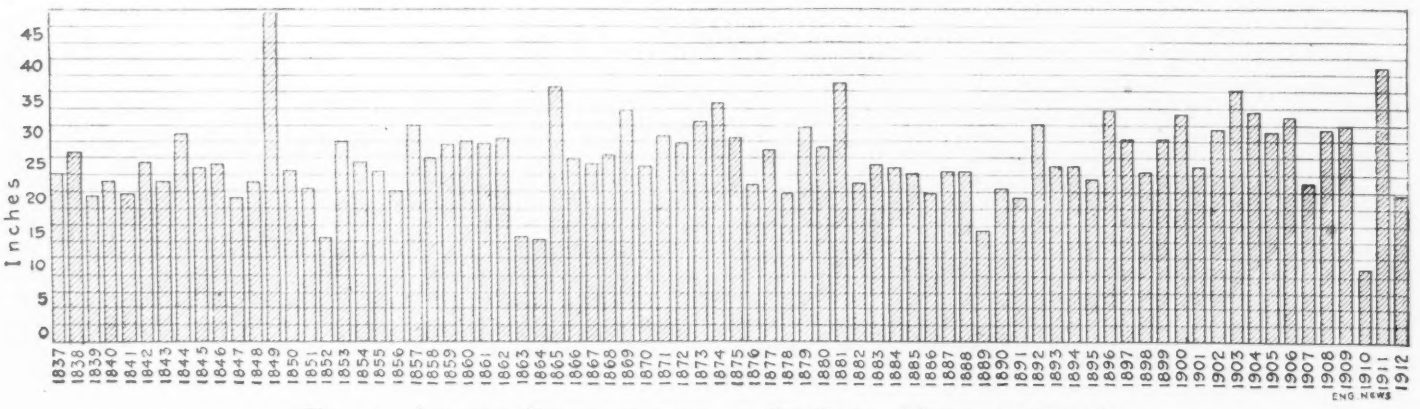


FIG. 1. ANNUAL PRECIPITATION AT ST. PAUL, MINN., 1837-1912

RECORD OF THE MONTHLY AND ANNUAL RAINFALL (IN INCHES) FROM 1837 TO 1912 INCLUSIVE, ST. PAUL, MINN.

Year	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total for Year
1837	0.27	0.35	0.33	0.95	2.65	3.46	2.73	1.32	5.10	3.15	1.37	2.34	24.02
1838	0.65	0.76	0.15	2.41	3.05	4.76	11.10	3.08	0.71	0.16	0.43	0.45	27.72
1839	1.34	0.36	0.71	2.71	3.28	1.80	3.50	1.04	1.61	2.11	1.66	1.07	21.19
1840	0.49	0.49	0.65	1.55	2.31	3.50	2.89	3.40	2.33	2.21	3.22	0.13	23.17
1841	0.24	0.21	1.43	1.40	1.50	4.24	1.57	1.17	6.10	1.55	0.84	1.42	21.67
1842	0.95	0.72	0.44	2.17	1.68	3.73	1.78	4.81	4.83	1.35	3.46	0.60	26.52
1843	1.15	1.46	0.82	0.75	3.12	5.22	2.09	1.84	5.14	0.50	1.43	0.27	23.79
1844	1.50	0.72	0.97	5.16	4.50	1.64	4.80	4.37	4.26	0.97	0.77	0.58	30.24
1845	0.49	1.40	2.80	3.15	1.51	6.80	2.56	3.28	2.21	0.66	0.40	0.08	25.34
1846	0.52	0.03	1.71	2.90	2.00	3.10	4.95	3.86	2.33	2.45	2.10	0.21	26.10
1847	0.29	0.11	0.44	0.45	4.96	2.66	3.66	2.49	4.00	0.37	1.71	0.66	21.80
1848	0.62	1.13	1.71	0.18	5.28	2.83	4.60	3.19	2.46	0.68	0.10	0.40	23.18
1849	1.00	0.61	4.11	5.62	6.57	3.14	7.59	9.60	2.75	5.35	1.40	1.95	49.69
1850	1.67	0.83	2.23	2.60	0.57	4.62	6.15	2.97	1.82	0.32	1.68	0.04	25.50
1851	0.20	0.13	1.23	2.68	3.96	2.15	2.60	3.29	3.61	1.18	2.31	0.05	23.39
1852	0.06	0.14	2.04	2.49	4.72	0.08	2.74	0.89	0.72	0.82	0.22	0.15	15.07
1853	0.00	0.01	0.02	0.73	4.08	7.59	1.65	2.57	2.14	0.01	0.56	1.11	29.47
1854	0.72	0.03	1.03	2.51	4.30	3.31	3.92	1.75	6.55	1.23	0.60	0.64	26.59
1855	1.67	0.41	1.87	0.28	1.23	2.38	1.32	4.41	6.26	0.90	2.38	1.67	24.78
1856	1.89	0.18	0.22	4.47	1.62	0.76	2.47	1.09	3.24	3.97	1.70	2.01	22.62
1857	3.48	0.94	0.79	4.25	2.05	6.74	0.65	2.03	2.46	0.00	5.75	2.95	32.09
1858	0.87	0.54	1.22	3.79	3.41	3.94	3.40	3.16	3.17	1.60	1.29	1.00	27.58
1859	0.60	0.50	3.33	1.70	4.95	5.49	2.95	2.72	3.63	0.46	2.01	0.75	29.09
1860	0.14	1.15	1.04	2.92	6.85	5.09	1.76	0.88	4.72	4.64	0.52	0.41	30.03
1861	0.35	1.94	1.15	2.50	6.34	4.82	2.69	2.97	2.80	2.54	0.10	29.93	
1862	1.35	0.59	0.95	2.67	1.35	2.66	10.15	4.58	2.89	1.06	0.59	1.37	30.21
1863	1.12	0.93	1.97	0.80	2.87	0.02	0.63	3.19	1.23	1.44	0.29	1.30	15.79
1864	0.38	0.80	1.28	0.56	0.47	1.62	4.00	2.00	1.14	1.80	1.09	0.71	15.65
1865	0.65	1.85	2.10	4.29	4.20	6.72	2.55	9.16	1.90	2.30	0.23	2.19	38.14
1866	2.09	0.30	1.32	2.60	0.39	6.06	3.59	4.73	2.26	2.64	3.19	0.50	28.23
1867	0.97	1.12	1.00	1.59	2.42	7.42	3.22	2.13	5.39	0.76	0.58	0.15	26.75
1868	3.25	2.74	1.40	1.26	3.17	1.50	2.52	2.29	2.21	3.68	2.80	1.25	28.07
1869	1.10	5.12	0.41	0.51	3.10	3.53	4.58	6.37	7.69	0.97	0.58	0.87	34.83
1870	3.09	1.11	2.78	1.11	3.32	0.81	2.63	5.86	2.49	1.39	1.13	0.35	26.07
1871	1.61	0.41	2.06	4.25	2.70	6.56	1.48	4.87	2.18	1.90	1.41	1.20	30.63
1872	0.28	0.20	1.64	1.69	5.71	3.84	4.23	3.52	5.62	0.52	1.91	0.63	29.82
1873	1.31	1.54	1.34	2.44	4.63	7.74	3.83	4.61	2.56	2.57	0.79	0.38	33.74
1874	0.49	1.07	2.24	0.95	1.65	11.67	1.94	3.90	5.76	3.21	1.90	0.72	35.50
1875	1.41	1.72	2.19	2.27	3.06	4.33	0.82	8.74	2.16	1.56	0.84	1.56	30.66
1876	0.73	0.66	1.43	2.23	3.15	2.02	2.73	5.28	2.99	1.27	0.93	0.25	23.67
1877	0.55	0.01	1.57	1.92	5.43	7.13	0.52	2.83	2.56	3.62	1.24	1.42	28.80
1878	1.90	0.67	1.24	2.43	2.53	3.58	4.47	1.43	1.13	1.83	0.61	1.04	22.78
1879	0.11	1.12	0.97	0.45	7.18	1.76	9.32	2.78	2.26	2.56	1.41	2.47	32.39
1880	0.81	0.97	2.30	1.58	2.63	5.55	2.75	3.63	1.74	2.18	2.93	2.69	29.76
1881	4.34	2.65	1.16	0.47	4.34	2.87	2.60	4.55	9.95	4.44	1.36	0.53	39.16
1882	0.67	2.35	3.25	1.61	2.44	2.68	2.84	2.45	0.27	1.99	1.61	0.98	23.14
1883	0.64	0.44	0.66	4.92	2.12	7.10	4.33	1.22	2.23	1.10	1.01	1.59	26.70
1884	0.48	1.27	1.34	2.00	2.09	3.57	2.93	2.89	4.48	2.43	0.65	1.98	26.11
1885	0.30	0.20	0.55	3.19	2.12	3.73	5.86	3.69	3.52	0.93	0.60	0.64	25.33
1886	1.76	0.25	1.09	3.67	0.82	3.63	1.44	2.27	3.69	0.72	2.07	1.48	22.89
1887	1.79	0.89	0.33	3.14	1.60	2.89	3.89	3.37	4.35	1.48	0.65	1.47	25.85
1888	0.72	0.64	1.11	5.14	4.75	1.95	5.55	2.23	1.70	1.10	0.34	0.63	25.86
1889	0.55	0.31	0.99	1.14	2.80	1.01	3.68	3.56	0.51	0.00	0.97	1.32	16.96
1890	0.95	0.50	1.11	1.80	3.66	5.29	1.87	2.20	2.73	2.79	0.38	0.10	23.38
1891	1.01	1.18	0.94	1.71	1.33	4.08	2.07	3.42	0.58	1.57	1.03	2.87	21.79
1892	0.02	1.44	0.75	0.97	5.17	7.50	9.04	3.66	1.72	1.39	0.36	0.53	32.55
1893	0.73	1.87	1.96	5.30	2.66	2.00	1.68	2.40	2.70	1.49	0.81	2.35	25.95
1894	1.04	0.10	3.28	4.30	6.63	1.51	0.13	0.36	1.82	4.49	0.61	1.53	25.89
1895	1.05	0.42	0.85	2.09	2.89	4.18	4.12	2.59	4.75	0.09	0.85	0.38	24.26
1896	0.81	0.17	2.93	5.63	4.67	3.30	1.12	4.48	2.45	3.39	5.07	0.71	34.73
1897	2.13	1.03	2.95	1.56	1.81	8.18	6.35	2.18	1.33	1.87	0.93	0.20	33.52
1898	0.04	1.69	1.96	1.22	3.40	2.71	1.94	3.93	0.90	5.81	1.59	0.15	25.34
1899	0.87	0.95	2.87	1.35	3.50	7.23	0.13	2.80	1.21	3.09	0.81	1.32	27.54
1900	0.66	0.73	1.48	1.80	0.34	1.98	5.81	3.13	9.39	7.55	0.88	0.47	34.22
1901	0.37	0.40	2.52	1.14	1.64	7.21	2.30	2.08	5.04	0.91	1.60	0.54	25.75
1902	0.53	0.68	0.58	3.04	3.92	2.49	5.50	5.64	3.16	1.33	2.40	2.48	31.75
1903	0.27	0.62	2.33	3.46	5.28	1.16	5.61	4.90	7.84	5.41	0.35	0.65	37.88
1904	0.41	0.76	1.47	1.74	3.20	4.99	5.73	5.41	4.56	5.05	0.12	0.73	34.11
1905	0.73	0.39	1.02	0.30	4.13	6.24	2.71	6.07	4.15	2.49	2.47	0.04	30.76
1906	1.20	0.22	1.07	2.23	10.40	4.55	2.57	1.83	4.71	1.91	1.73	0.79	33.21
1907	0.81	0.83	0.53	1.32	1.27	4.53	4.70	4.07	2.48	0.96	1.08	0.49	23.07
1908	0.52	0.87	1.16	4.23	7.57	5.14	2.94	1.07	3.96	2.19	0.67	1.28	31.60
1909	1.56	2.18	0.49	2.42	3.37	4.41	3.66	2.89	4.06	2.60	2.66	1.52	31.82
1910	1.10	0.42	0.16	0.59	1.76	0.91	0.99	0.98	1.77	0.75	0.48	0.30	10.21
1911	0.73	0.89	0.74	2.22	3.99	5.79	5.15	5.51	5.27	7.55	1.12	1.51	40.44
1912	0.52	0.07	0.32	2.60	4.06	1.10	4.02	4.40	1.27	1.18	0.08	1.54	21.16
Totals	71.70	65.72	104.88	174.22	256.06	304.68	264.83	256.24	250.47	156.63	102.24	75.22	2,082.90
Average	0.94	0.86	1.38	2.29	3.37	4.01	3.48	3.37	3.30	2.06	1.35	.99	27.41

AUTHORITIES—
1837-1855—Recorded at Fort Snelling.
1856-1859—Minneapolis, by Dr. C. L. Anderson.
1860-1864—St. Paul, by Rev. A. B. Patterson.
1865-1870—Minneapolis, by Mr. Wm. Cheney.
1871-1912—St. Paul, United States Weather Bureau.

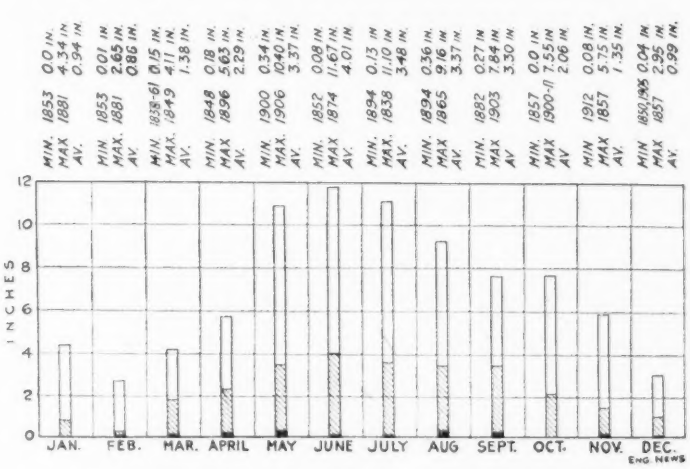


FIG. 2. MONTHLY VARIATION IN PRECIPITATION AT ST. PAUL, MINN.; FROM RECORDS 1837-1912

son of the mean annual rainfalls. In the 75-year period there are two months when absolutely no rainfall was recorded, January, 1853, and October, 1857.

The rainfall in the eastern part of Minnesota is heavier than the western part. The mean annual rainfall for the state is about 2 in. less than that for St. Paul.

A New Device for Locating Leaks in Water Mains

The Department of Water Supply of New York City has recently experimented with, and purchased, a new device for detecting and locating underground leaks in water mains, which is known as the "Pulsograph." This apparatus has been designed and built by N. W. Akimoff, a hydraulic engineer of Philadelphia, Penn., who, at our request, has furnished a brief description of the principles underlying the operation of his apparatus.

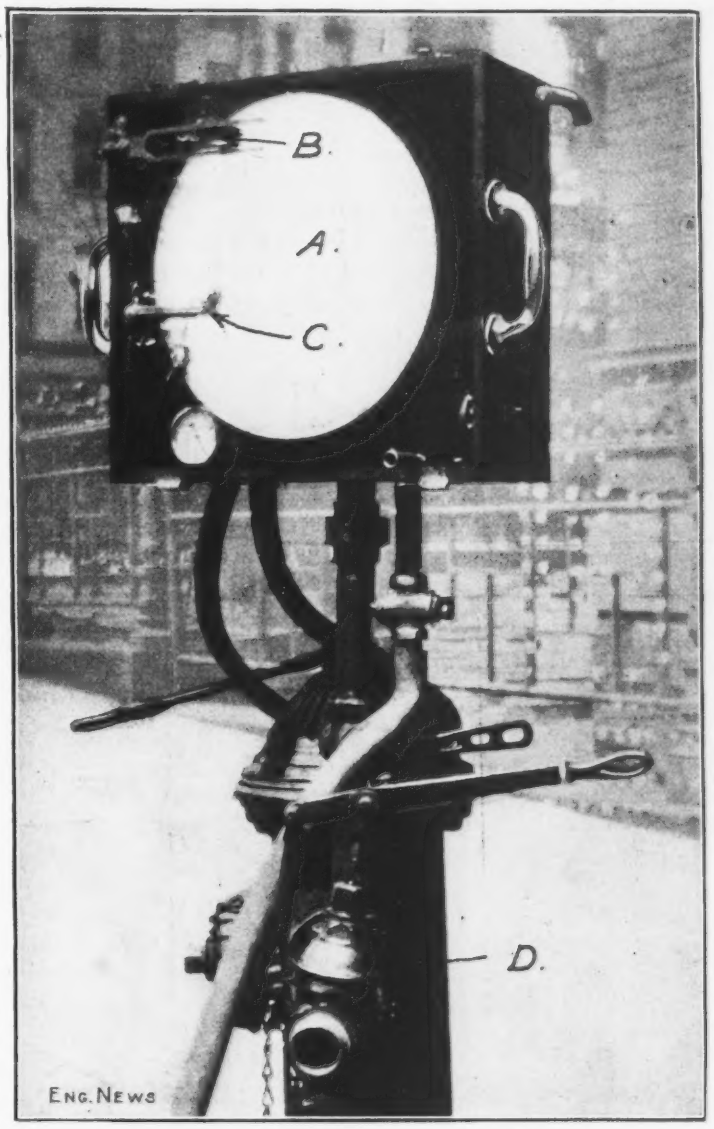
According to Mr. Akimoff, Prof. Joukovsky, of Moscow, Russia, in his well known investigation of the nature of water-hammer, was the first to prophesy the possibility of applying the theory of the propogation of the shock wave to the practice of leak location. The theory of water-hammer has been investigated by Joukovsky (1897), Allievi (1904), Boussinesq (1905), Flamant (1906) and by Prof. Irving P. Church, of Cornell University, and two or three other engineers in this country. The deductions of these investigators agree fairly well with the results of tests with the pulsograph, so far as the propagation of the shock wave is concerned.

The pulsograph registers the time elapsed between the beginning of the shock, which is caused by quickly closing a small quick-lever valve, placed together with the apparatus on an ordinary hydrant (at the end of the block which it is desired to investigate), and the drop of the shock pressure due to the leak, which may be encountered by the shock wave or pulsation in its travel upstream. If there is no leak, the pulsation dies out on reaching the upper end of the block, or, in general, an intersection with another main, as the case may be.

Only a mild shock is required for operating the pulsograph and the speed of propagation of the puplsation is independent of the extent of the shock, and travels along the pipe at a velocity of about 4000 ft. per sec. The pulsograph is so proportioned that the lawful use of water by consumers does not interfere with its operation; so tests may be made in daylight and without shutting off any services. Mr. Akimoff admits that the apparatus has its limitations, but that when applied with care good results can be had.

Regarding the use of the pulsograph in New York City, W. W. Brush, Deputy Chief Engineer of the Department of Water Supply, Gas and Electricity, in a paper entitled "Notes on Reduction of Waste of Water in New York City," read at the Philadelphia convention of the New England Water Works Association, Sept. 10, had this to say:

> The pulsograph consists essentially of a large sensitive pressure recorder (C) having a rapid motion, a tuning fork (B) vibrating at a rate of 200 vibrations per sec., and a quick operating valve (D), which is on a blow-off or by-pass pipe.
>
> The instrument is set up by screwing it to a hydrant nozzle, the hydrant being chosen so that there will be a straight run of several hundred feet before an open four-way branch is encountered. By closing valves on intermediate branches, the effect of a long, isolated line can be obtained.
>
> In operation, the valve on the main below the hydrant is closed and water allowed to flow from the hydrant through the by-pass on the instrument. By quickly closing the by-pass valve a water hammer of about 15 lb. is created and recorded on the pressure gage. This pressure is maintained until the water hammer pressure has reached the open branch at the end of the pipe line that is being tested and the drop in pressure has traveled back to the hydrant; the tuning fork vibrations showing the time for this double travel.
>
> By dividing twice the distance along the main to the open branch by the time, the rate of travel for the particular pipe is determined. This has been found to be approximately 3600 ft. per sec. for 6-in. pipe and increasing to about 4200 ft. for 12-in. pipe.
>
> If there is any leak in the main, or in a service near the main, the water-hammer pressure is materially reduced by the leak, and the chart shows separately the reduction due both to the leak and the open branch.
>
> The distance to the leak can then be determined by the time shown on the chart for the water-hammer pressure to travel to the leak and return to the pulsograph, this being multiplied by the determined rate of travel of the hammer pressure along the pipe, and divided by two.
>
> Several leaks have been located within limits of less than 20 ft. by the use of this instrument, and the results from its further use should be interesting, as a decided saving in cost of locating leaks is anticipated.



PULSOGRAPH IN PLACE ON HYDRANT

A Recording dial.
B Time-recording vibrator.
C Pressure recorder.
D Quick-operating valve to create water hammer.

We are informed that the pulsograph is not for general sale, owing to the rather intricate nature of computations in calculating the results and interpreting the records.

Valuation of Buildings

By Robert H. Overstreet*

The valuation of buildings is one of the important divisions of the work of appraisal of public-service properties, as the investment in that class of physical property will in most cases amount to from 4 to 8% of the total capital invested; contingent, of course, upon the character of service that the utility renders.

At the outset and where it is practicable, a reconnoissance of the property ought to be made, in order to determine the most advantageous starting point. Naturally, it would not be economical or feasible to do this if the property is distributed throughout the country at great intervals.

After this general review of the property the quantities may be taken off from the available plans and specifications. For this work the accompanying form has been found very serviceable in the arrangement of items, and shows the evolution of the work from the excavation to the intricate details of plumbing, lighting, etc.

BUILDING SUMMARY

Name of building..... Sheet No.....
Location..... Client Figured by.....
Date of valuation..... Priced by.....

Classification	Quantity	Unit cost	Total cost	Remarks
Excavation				
Piling				
Concrete				
Masonry:				
Brickwork				
Cut stone				
Rubblestone				
Tile				
Steelwork:				
Structural				
Reinforcing				
Steel sash				
Carpentry and joinery:				
Doors				
Windows				
Lumber				
Plastering				
Ornamental Ironwork				
Sheet metal work				
Roofing				
Hardware				
Painting				
Plumbing and fixtures				
Heating and ventilating				
Lighting				
Miscellaneous				
Total material and labor				
Contractor's profit, %				
Total cost of actual construction				

The original sheet is 8½x11 in., for binding in a letter-sized loose-leaf book.

The first item under the classification is "Excavation." In figuring the quantity of material removed always allow at least 18 in. on all sides in addition to the actual dimensions. This is done in order to allow for a reasonable space between the wall and earth so that a mason can work there with ease. Excavation for machine foundations should not be included in the building quantity, as it is chargeable to that particular branch of equipment to which it serves.

On account of location and size, some buildings require piling. When the foundation plans do not show the lengths, the number of linear feet may be estimated by ascertaining from contractors the usual depth to which piling is driven in the vicinity; the number of piles may be figured by consulting the city specifications on centering.

Little trouble should be met in figuring the concrete work, which ought to be apportioned according to its use in the building, i.e.; foundations, walls, columns, floor slabs, etc. Concrete ought never to be tabulated as reinforced, as anyone checking over the building data would not know whether the reinforcement were a patent type or plain reinforcing rods. The concrete and reinforcement should be priced separately.

In the matter of brickwork, the following table of allowances has been found to be a very correct method for obtaining the number of bricks. As bricks are purchased by the thousand it is logical that they should be figured in the wall at the same unit rather than by the cubical contents. As shown by the following table, allow 7½ bricks for every additional 4-in. thickness of wall:

Thickness of wall, inches	Thickness of wall, bricks	Bricks per superficial foot
5	0.5	7.5
9	1.0	15.0
13	1.5	22.5
17	2.0	30.0
21	2.5	37.5

In taking down the lengths of the walls consider them over their longest points. This will necessarily duplicate the contents of the corners but will compensate for the waste in cutting the brick for intersections and will also cover breakage. Make a deduction for 50% of all door and window openings. Other items under masonry such as cut stone, rubblestone and tile, can be figured with little difficulty, the unit quantities being cu.ft., cu.yd. and sq.ft., respectively.

As a general rule the plans for the structural steel of a building are very accurate and require little checking, as the fabricating mills must have the exact dimensions in order to make the proper assemblage. In latticed columns, trusses, etc., a percentage is added for rivets, but rather than add a percentage to the total weight of all the steel in the building, the writer has found that more accurate results are obtained when the ratio of the weight of rivets to each particular member is found and the percentage thus deduced added to the weights of all members of that type. The weights of each particular section may be found in any steel handbook. The following form may be used conjunctively for steelwork and lumber and if an error has been made in taking off the quantities from the plans it may easily be rectified:

ESTIMATE OF { Steel work / Lumber

Name of building..... Sheet No.....
Location..... Client Refers to drwg. No.
Date figured..... Figured by.....
Checked by .....

Description	No. pcs.	Length	Weight in lb. per ft.	Total weight	Total ft. b.m.	Remarks

The original sheet is 8½x11 in., for binding in a letter-sized loose-leaf book.

If the steel sash are of standard make, they may be priced from the catalog. The dimensions, number of lights in each sash and manufacturer's name should be specified. If no catalog can be found to cover the particular type of sash, price them on the basis of the number of square feet they contain.

Doors and windows probably are the most difficult part of a building to figure. An inventory of the doors should always specify the exact dimensions, the number of panels and the kind of mouldings. If a transom door, the

*730 Hoyt St., Portland, Ore.

height should be given as well as the number of lights; the width is that of the door.

In regard to windows, obtain the three dimensions of each sash, specify whether check or plain rail, whether single or double hung and the number and size of lights in each. Frames and trim should be described with their respective doors and windows. It is obvious that all diversions from the standard should be noted and for the purposes of this paper it will not be necessary to go into detail regarding special types of construction, etc.

If bills of material are used in estimating lumber, it is preferable to take the length of the piece to the next even foot. If, e.g., the plans show a joist to be 16 ft. 6 in. long, tabulate the length as 18 ft., as the latter length would have to be purchased. If the actual lengths are taken a percentage should be added to the total number of feet, B.M., to allow for waste. In ordinary framing this percentage would not be greater than 3½.

Plastering should be specified as to the number of coats, whether or not it is on metal or wood lath or masonry and should be figured by the square yard. The walls should be figured as free from openings, with the exception that any opening over 150 sq.ft. should be deducted as well as wainscoting that is not plastered behind.

Sheet-metal work is not difficult to figure. For flashing figure the number of square feet as shown on the plans and obtain the size and number of linear feet of downspout. The measurements of sky lights should be taken as well as the manufacturer's name, that they may be priced by referring to the catalog. If not of a standard make they may be priced by the square foot, depending upon whether they are flat or hipped.

Roofing is figured by the square, i.e., 100 sq.ft. If a patent roofing, the number of ply should be specified as well as the character of the roof that it is laid upon. It costs more to lay roofing on a concrete base than it does to lay it on one constructed of wood. Shingle roofs are sometimes figured by the number of shingles used. If the former method is adhered to, it is necessary to specify the amount of exposure of the shingles or, to use the trade term, "their distance to the weather."

If the quantity of hardware that has entered into the construction of the building is not known, the number of pounds may be estimated by consulting any table of nail allowances for the particular class of work.

When possible the contract prices on painting, plumbing, heating and ventilating, lighting, etc., should be obtained, as this work is usually let to contract and can be figured more closely by persons engaged in that class of work.

Under the heading "Miscellaneous," classify all items which do not logically come under the other headings.

There are many special classes of construction and of materials which will not conform exactly to the foregoing method of procedure. For such, the method will require more or less readjustment.

Pricing

After bills of material have been taken, the next important step is to apply unit prices to each group of quantities. In case the building has been of such size that it has consumed several years in construction, it would undoubtedly be more accurate to average the commodity and labor prices over that period. In a building, the structural steel would be the material most liable to fluctuate and its price should be averaged over the construction period, using the prices in effect at the date of the valuation for the other materials.

The units should include only the cost of the material laid down at the building site and the actual labor necessary in the erection thereof, as a contractor's profit is added to the total cost of the material and labor.

Each unit cost should be analyzed and the following is offered to suggest one of the methods. The prices are arbitrary and the amount of work accomplished and the proportions of materials are contingent upon the character of the building and the specifications respectively. The following is an analysis of the cost of 1000 brick in the wall with cement mortar, assuming that one mason in an 8-hr. day lays 1000 brick:

1000 brick delivered at site	$8.00
One mason (8-hr. day)	5.00
One laborer (8-hr. day)	2.00
1¼ bbl. cement at $1.35 per bbl.	2.03
¼ yd. sand @ 70c. yd.	0.44
Water	0.04
Total price (per M)	$17.51

In like manner the costs of other materials may be analyzed, and accurate and detailed accounts of all prices are thereby at hand. Should there be any question, by the clients, regarding the fairness of the units the analyses may be submitted for verification.

A contractor's profit should be added to the total cost of the material and labor. Where contract prices are used, care should be taken not to include those items in the total to which the percentage has already been added. As a rule 10% is the amount allowed for profit, but in some cases it is as high as 15%.

In addition to this percentage there are certain overhead charges that rightfully should be applied to the actual construction cost. These charges can be determined only by an examination of the records of the company. Engineering and supervision is usually a fixed charge of 5% and includes the cost of all preliminary work, such as drawing up plans, laying out properties, as well as supervision of the construction work. The other items: legal and organization expenses; interest during construction; taxes and insurance, and contingencies and omissions should be added to the total actual cost of the entire property and not to each separate building or structure. Adding these percentages gives the cost of reproduction new.

Depreciation

Having found the "Cost to Reproduce New," the question of depreciation presents itself. There are several methods of handling this, but the one most in use is the "Sinking Fund Method." It is a fact that a new machine just installed or a building just constructed does not submit to the wear and tear and does not deteriorate as rapidly at first as it does after some time in service; it is for this reason that the above mentioned method is used. A useful life, suitable to the type of construction of the building is assumed and the percentage of depreciation or condition per cent. is determined, based upon the assumption that the company invests a certain amount each year which when compounded annually at 4% (arbitrary) equals the initial cost of the building at the termination of its usefulness.

Tables showing the per cent. conditions for the average interest rates are to be found in a number of technical books, but if the table is not at hand, substitutions in the following formula will give the desired results. Suppose a brick building has been in service 10 yr. and its life is assumed to be 50 yr. and the rate of interest that a fund is capable of earning is 4%. Then let i = interest rate; L = the assumed life of the building in years; a = the age or number of years in service; $(1 + i)$ = compound interest. Then by substituting in the formula,

$$\textit{Per cent. of depreciation} = 100\left[\frac{(1+i)^a - 1}{(1+i)^L - 1}\right]$$

$$= 100\left[\frac{(1+.04)^{10} - 1}{(1+.04)^{50} - 1}\right] = 7.86 \textit{ or } 8\%$$

$$\textit{Condition per cent.} = 100 - 100\left[\frac{(1+i)^a - 1}{(1+i)^L - 1}\right]$$

or in the example under consideration, $100 - 8 = 92\%$.

Hence, the present value of the building is 92% of the reproduction cost.

The above method more closely coincides to the actual physical depreciation than does the straight-line method, used quite extensively. This latter method is based upon a simple ratio of the age to the assumed life. Suppose that a building constructed of brick and steel has a life of 50 yr. and at the present time is 10 yr. old; then the present value according to the straight-line method would be $\frac{10}{50} = \frac{1}{5} = 20\%$ deducted from the reproduction cost.

It will be seen that no account has been taken of salvage value. In a building practically the only thing salvageable is the steel, and unless it were a building of great size, it is doubtful whether there would be much profit after the cost of razing had been deducted. In many cases it costs about as much to tear down as the material will bring in salvage value; hence this item has been considered as practically *nil* in regard to buildings.

NOTES FROM THE ENGINEERING SCHOOLS

Massachusetts Institute of Technology—A committee of the Alumni Council has recommended that a new four-year undergraduate course should be arranged, for fundamental training in science and engineering together with instructions in business methods, business economics and law, to be designated as "Engineering and Business Administration." The curriculum as outlined tentatively, is arranged for some degree of specialization along lines of mechanical, chemical and civil engineering, with the object of fitting men to engage in (1) the manufacture of machines and machine products, (2) the manufacture of materials, or (3) transportation and construction work. The business studies proposed would include political economy, economic history and geography, accounting and statistics, commercial organization, business law, banking and finance, industrial psychology, argumentation and reports, taxation and insurance, transportation and foreign trade; it is recommended that these studies should not be taught for the most part by formal lectures, but by requiring the student to study specific cases and problems. The committee also urges coöperation with the School of Business Administration of Harvard University, if practicable. The business studies included in the proposed under-graduate course would be made available for graduates of the regular engineering courses who desire to supplement their work by a fifth year; conversely, the graduates of the new course would have an opportunity to study engineering further in an additional year.

⁂

Coöperation for Protection from Forest Fires—During the last quarter of the fiscal year, the federal government entered into coöperative fire protection agreements with the following states: Maine, New York, Minnesota, Montana, Washington and Oregon. These six agreements contemplate the protection from fire of approximately 87,000,000 acres of land on the forested watersheds of navigable streams, for which purpose $51,500 of federal funds have been made available. There are at the present time 14 states which are engaged in active coöperation of this kind.

The federal government, though it has allotted a total of $85,000 for the work of the present season, is a minor contributor in the aggregate. Under the impetus given to fire protection of the watersheds of navigable streams through this arrangement the coöperating states will expend an aggregate of $5 for every dollar which the federal government spends, to which may be added about $2 more from private sources.

These amounts, of course, do not represent the sum total of expenditures for fire protection even in these states. For in some of them the state itself is spending considerably more money in the protection of forested lands lying outside the watersheds of navigable streams. On the national forests, too, the government is spending several times the amount of its contribution to coöperative fire protection in the states which contain federal timberland.

Aside from these governmental agencies the railroads, lumber companies (both individually and through their cooperative protective associations), municipalities and private land owners throughout the country are growing more and more alive to the needs of better forest fire protection and are devoting each year an increased sum to this work.—From U. S. Forest Service.

A 33-Mile Submarine Telephone Cable has recently been laid in the Gulf of Georgia between Vancouver Island and British Columbia by the W. T. Henley Telegraph Works of London. From a paper by E. P. La Belle and L. P. Crim before the American Institute of Electrical Engineers, Sept. 9, 1913, it appears that this cable was to provide better facilities for Vancouver Island with a speaking range from any point on the island to Vancouver and the other principal towns on the mainland.

In the cable are four conductors, each consisting of 13 small copper wires weighing together 300 lb. per nautical mile. Around this is a soft-iron wire (0.012-in. diam.) wound 70 turns per inch. This "loaded" conductor is covered with three coats of gutta-percha (300 lb. per nautical mile). The four conductors are wound around a center yarn with yarn worming between them. The whole is covered with brass tape, yarn and an armor of 15 galvanized-steel wires (0.192-in. diam.). This is served with yarn, tarred and covered by two layers of yarn and preservative. The completed cable has a diameter of about 2 in. and weighs 17,900 lb. per nautical mile. The electrical circuit constants (for two conductors per nautical mile in place) are resistance 8 ohms, mutual capacity 0.175 microfarad, insulation resistance 460 megohms. The four wires are worked with a "phantom" circuit so that three simultaneous conversations are carried on. The phantom circuit, however, is much inferior to the two physical circuits through having an electrical capacity twice as great. Speech tests on the cable in place showed an equivalent of 8 miles of standard land cable (No. 12 paper-insulated lead-covered) connected by a zero loop at each end, and of 5.75 miles with 12 miles of standard cable at each end to prevent reflection impulses.

This is the only cable of the kind in America; on account of the depth to which it had to be submerged (1500 ft.), gutta-percha insulation was used and on account of the excessive electrical capacity it was necessary for good speech transmission to balance this with added inductance or "loading." The continuous loading with iron wire was adopted in place of the insertion of regular Pupin coils at intervals on account of the depth of submersion and the ease of repair. The cable was laid in one day (June 16, 1913) between 4 a.m. and 7:30 p.m. According to the London "Electrician," of Aug. 22, 1913, the total cost was about $120,000.

EDITORIALS

The scheme to make the federal memorial to Abraham Lincoln take the form of a grand boulevard from Washington to Gettysburg or to Baltimore having been foiled in Congress not long ago, the same or other interests have organized the Lincoln Highway Association to build an Atlantic-Pacific Highway. It is reported that several million dollars have been subscribed by automobile manufacturers, that automobile users are being solicited to buy certificates and radiator name plates to aid the cause and that the cement manufacturers have promised to give 0.5% of their output for three years to aid the project. What truth there may be in these reports we do not know. The reported method of financing the scheme has the merit of placing the burden on those chiefly benefited, but what would Abraham Lincoln say to the use of his name for a project designed so largely to foster purely commercial and pleasure-chasing interests and for a public improvement no more lasting than the road surfacing of the present day?

Stronger Buffers Needed for Railroad Terminals

The value of adequate buffers in railroad terminals will be better appreciated since the two accidents which took place recently. The first occurred on June 15 at the Place Viger station of the Canadian Pacific Ry., at Montreal, Que. A flat-car bearing a tank filled with coal tar while being shunted into the Mile End yard escaped from its engine and ran down the grade into the station, carrying away the bumpers. It crashed into the steel gates, separating the track termini from the concourse, ploughed its way across the concrete concourse and through the side of the building, on into the waiting-room where many were awaiting the arrival of the night trains. The brick walls crumbled from the impact and a reinforced-concrete column extending from the floor to the roof was badly twisted. From there the car tore into a bank of seats and came to a stop against another column, where it turned upon its side spilling the load of tar.

The other accident occurred at the Erie R.R. terminal, Jersey City, N. J., Aug. 2. The Suffern express was backing on Track 6 preparatory to taking on her passengers. When the rear of the seven-coach train was about 200 ft. from the gate shutting off the long platform from the tracks, the rear car couplings broke loose. The train was immediately stopped with the exception of the last car which continued on. The bumpers were demolished as were the gates and the announcing board. The roof of the car demolished the bridge running along the second story of the passenger station and overlooking the tracks. The wheels of the car broke through the 3-in. plank flooring of the wooden concourse and finally stopped at the entrance to the waiting-room.

In another part of this issue we print an abstract of a paper which outlines a set of tests which describe the performance of a certain hydraulic buffer installed in the Grand Central Terminal, New York City. The conclusions arrived at through these tests were to the effect that the hydraulic buffer of the design installed experimentally, safeguarded the equipment under the conditions of train speed and train weight imposed during the tests. Whether further improvements in design will be found necessary before this buffer can be pronounced entirely acceptable and be installed generally to supersede those at present in use is a matter for further investigation, and the settlement of the question will be received with interest.

Accidents of the character of the two just described, with their dangerous and gruesome possibilities added to the damage done to property, should have the effect of making a more efficient form of bumper or buffer welcome if not imperative. The common form of bumper is evidently not designed to stand more than a comparatively gentle impact. The expense entailed by the installation of a strong type of protective device in railroad terminals for preventing cars from overrunning the track ends will prove a paying investment in the long run.

A Remarkable Professional Fee

Youths who have been fired with the determination to become civil engineers by reading newspaper stories concerning the huge earnings of eminent engineers, may now have their ambitions directed toward a new channel. According to the press reports of court proceedings at Nyack, N. Y., last week, the most highly paid professional man is not a consulting engineer, nor even a lawyer. He is a "Consulting Contractor."

Before Judge Tompkins, of the New York Supreme Court, Harry B. Hanger, of the Mason & Hanger Co., contractors, of Richmond, Ky., testified, under cross examination, that he had paid a fee of $30,000 to James E. Gaffney, of New York City, as a fee for his expert advice on the procedure necessary to obtain the award of a contract for a section of the new Catskill Aqueduct work.

The suit in which testimony was given is one brought by James D. Mundy, a partner of Hanger in the Mason & Hanger Co., for a receivership and accounting. Under cross examination, Mr. Hanger further testified that the $30,000 had been paid to Gaffney in two installments of $15,000 each, and in currency and not in checks; that the payment had been entered on the books of the contracting company under "legal services," without mention of the party to whom the payment was made. When pressed for the specific advice which he received from Mr. Gaffney in return for this fee, Mr. Hanger said that he conferred with him about the labor laws and especially the union labor situation, and that he asked Gaffney to see the labor unions and tell them that the Mason & Hanger Co. would recognize union labor on the work.

At the time the advice was obtained and the fee paid, the Mason & Hanger Co. was the lowest bidder for the

work on Section 20 of the Aqueduct, but the award of the contract had not been made. After the $30,000 payment, the Mason & Hanger Co. received the contract. Mr. Gaffney himself is said to be in the contracting business as a partner in the Bradley-Gaffney-Steers Co. He has also achieved fame by his political affiliations, and was said to have been recommended by his backers to Governor Sulzer last spring for appointment as Commissioner in charge of State Highway Construction in New York. He was not appointed.

We presume this fee paid to Mr. Gaffney will hold the record for a long time as the largest professional fee paid for the smallest amount of advice. If any other "Consulting Contractor" wishes to report to us authentic information regarding the receipt of a larger consulting fee, or if Mr. Gaffney himself desires to report the receipt of a larger sum from some client not now publicly known, we should be glad to place the facts on record in the pages of ENGINEERING NEWS.

The Latest and Worst New Haven Railway Disaster

In all the annals of American railroading one would search far to find a railway company which has come under such severe public condemnation, chiefly as the result of fatal accidents occurring on its lines, as has the New York, New Haven & Hartford R.R. during the past two years.

Whatever the technical explanation offered respecting the causes of these accidents, the public has become convinced that the ultimate responsibility for them lies with the men at the top and not with the employees or the minor officials.

We have hitherto expressed the opinion that in some of the previous accidents this popular verdict has been unjust; but with respect to the disastrous rear collision of Sept. 2, it must be said that the chief responsibility lies, not with the members of the train crew whose acts directly produced the collision, but with those who have permitted a defective and wholly inadequate block-signal system, installed many years ago, to continue in use long after it should have been replaced by modern apparatus.

It is on this single cardinal defect in connection with this New Haven disaster that public attention should be concentrated in order to reap the greatest results in improvements looking to the safety of railway travel. In no other possible way can our shocking American record of fatal railway collisions be reduced save by reforming our practice in signaling.

The substitution of steel cars for wooden cars is in comparison of trifling importance. The way to prevent deaths in collisions is to provide means for train operation which will prevent collisions. The railways of England and Germany do it and have done it for many years. We must raise our standards of railway-signaling practice to equal those of Europe.

The important facts of the New Haven collision were briefly noted on p. 484 of our last issue and may be again summarized as follows: The collision occurred on the morning of Sept. 2 on the line between Springfield, Mass., and New Haven, Conn., at a point between Wallingford and North Haven, some 10 miles north of New Haven, and near the end of a 3-mile tangent. The close of the summer vacation season, marked by the Labor Day holiday just preceding, was crowding the road with a heavy volume of traffic, making it necessary to run trains in several sections.

The testimony before the Interstate Commerce Commission shows that six southbound trains passed Wallingford during 32 minutes just prior to the collision or an average time interval of barely 5 minutes.

One of these trains, the second section of the Bar Harbor express, was stopped just before reaching block signal No. 23. After standing three or four minutes, it slowly drew past the signal and stopped again for the rear flagman who had gone back. While standing there with its rear end some two car lengths past the signal, it was struck by the White Mountain express following, running at such speed that the two Pullman sleepers at the rear were telescoped and the third car was turned on its side. The resulting casualties were 21 persons killed outright while 5 received injuries probably fatal, a dozen others were seriously hurt, and 50 or more persons received minor injuries. In view of the extensive destruction wrought in the last two sleeping cars by the locomotive as it ploughed its way through them, and the fact that on this night of heavy travel nearly every sleeping-car berth is engaged, the only wonder is that a much larger number of deaths was not recorded.

The accident occurred at 6:51 a.m. in broad daylight, but a fog prevailed making it impossible to see signals more than 200 ft. or so away.

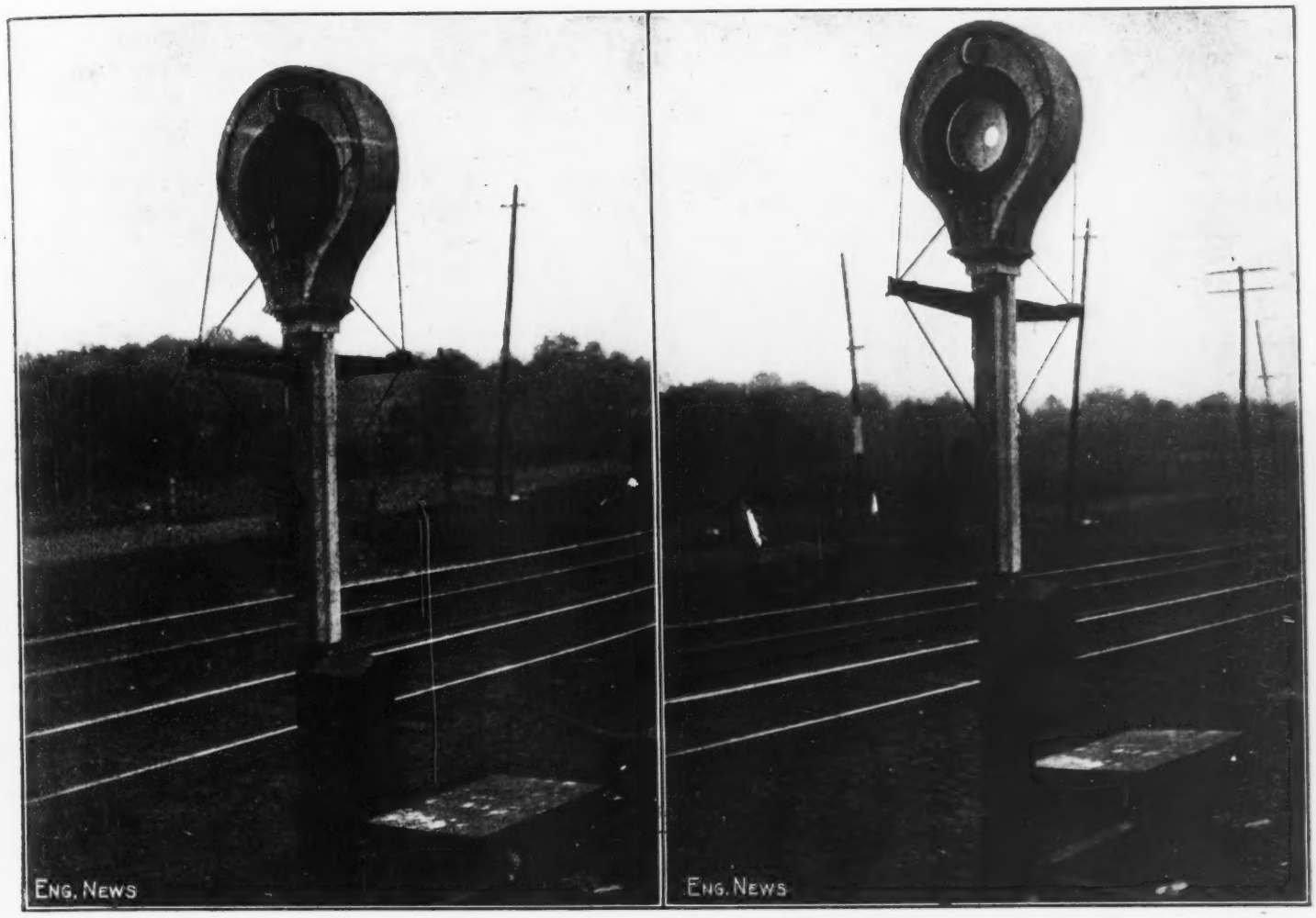
While the line of the New Haven road from New York to New Haven is protected by semaphore block signals manually operated, the line from New Haven to Springfield, over which many fast trains run, is protected by automatic-block signals of the inclosed-disk type, installed something like a quarter century ago and so far as at present appears never altered since in any way to conform to modern practice in safe signaling.

We say "protected"; but it did not require the horrible massacre in the North Haven collision to prove to anyone with even an elementary knowledge of railway signaling that the automatic signals on this line were a most defective and dangerous means of spacing trains.

Their cardinal defect was *the absence of anything in the way of a distant or caution signal in advance of the home signal.* Out of some 150 home signals between New Haven and Springfield only about 25 have distant signals connected.

For the benefit of readers unfamiliar with signal practice, we may explain that a "distant" signal (also called an "advance" or "caution" signal) is a signal located ahead of the home signal which indicates to an engineer the position in which the home signal is standing. If the home signal stands at "danger," the distant signal stands at "caution"; and when the engineer sees the distant signal in this position he knows that he must apply the brakes so that he can stop his train before he reaches the home signal.

In fact, in good modern signal practice, *the distant signal is the most important signal for the engineer to read correctly.* The home signal is really a marker fixing the limit within which he must stop his train. If he waits till he can see whether the home signal is at danger before checking his train, it may then be too late to stop before reaching it.



BANJO SIGNAL NO. 23, JUST BEYOND WHICH THE COLLISION OCCURRED; AT "DANGER" AND AT "CLEAR"

In block signals which are operated by men in signal towers, the distant signal is placed usually 1200 to 1800 ft. back of the home signal. In automatic-signal systems (in which the signal is operated by the train itself, by means of an electric current) the distant signal is placed beside the home signal next preceding, so that the engineer is apprised as to the position of the next home signal a whole block before he reaches it. Or a single signal may have three positions indicating respectively "safety," "caution" and "danger."

No automatic signals such as were in use on the New Haven are installed nowadays without distant signals; and few automatic signals without distant signals, we believe, are in use. If such there are, the terrible warning of the North Haven wreck should lead to their replacement at the earliest possible moment.

The New Haven block signals were further defective in that they were of the banjo or disk type instead of the semaphore type. The best railway practice the world over has fixed on the semaphore, or projecting arm standing out from a vertical post as the best type of railway signal. It can be seen and identified at a greater distance than the disk or banjo signal, used on the New Haven. It may be explained that the latter gives its indication by moving a disk of thin silk in front of a glass-covered opening in the face of a banjo-shaped case mounted on a pole. White indicates safety and a red disk danger.

A quarter century ago introducers of the banjo signal made much of the theory that since by night a signal's indications depend on color, the daylight indications should also be by color. But practice on more than a hundred thousand miles of railway the world over for nearly half a century has proved that the standard semaphore signal, with strong lenses aiding the lamps for night indications, gives almost perfect results.

VIEW OF SIGNAL NO. 23 FROM POINT ON TRACK, WHERE ITS OUTLINE IS SUPPOSED TO HAVE BEEN FIRST SEEN BY ENGINEER OF WHITE MOUNTAIN EXPRESS

Such a signal system as is in use on the New Haven railroad between New Haven and Springfield would not be permitted for a single day on a railway in Great Britain, where all railway operation is under the strict supervision of the Board of Trade. Nor would any German railway dare to use such a system.

It must be regretfully said that the existence of such an archaic and dangerous signal system on one of the main lines of the New Haven system goes far to confirm the most damning criticisms which have been made of the New Haven management.

It has been brought out in the testimony in the Interstate Commerce investigation of the recent accident that trains on this division frequently drifted past home signals at danger before coming to a stop; but with the signal system in use this was inevitable. The only way engineers could avoid doing this would be to approach every signal at a speed low enough to stop before reaching it, if found at danger. Except in the clearest weather this would mean slackening speed at every mile, if trains were to be moved over the road at the speed called for by the schedule.

On the morning of the fatal disaster a heavy fog obscured the signals so that an engineer could not read them until he was within 100 to 300 ft. Yet trains were being moved over the division at speeds reaching and probably exceeding 50 miles per hour, and with barely a five-minute time interval. It is difficult to fitly characterize such railroading.

A great deal is being made in the investigation of the questions whether the rear flagman went back far enough and soon enough and did all that the rules prescribed to stop the train following. But all the discussion over this matter tends to obscure the real lessons of the disaster. If our block-signal systems and the observance of block signals by employees were what they should be and what they are in Europe, we could dispense with back flagging altogether, and would probably make a gain in safety by so doing.

It is true that American railways in adopting block signaling have retained the rules for back flagging in use under the train dispatcher system. It is also true that these rules are not lived up to and that their observance in either letter or spirit is impossible on roads running fast trains closely spaced. Imagine the delays on a crowded road if every time a train was held up by a block signal the rear flagman had to travel half a mile to the rear and return to the train before it could again proceed. The only alternative would be to carry a carload of rear brakemen, and leave them behind, one after another, whenever the train came to a stop. Even then there might be trouble if the supply of brakemen ran out before the train finished its run.

The most important thing to bear in mind, however, is that the back-flagging system, even when carried out literally and conscientiously, leaves many loopholes of serious danger. A train may come to a stop and send a flagman back to the rear and the following train may run into it before the flagman can run back far enough to stop the approaching train in time.

According to the testimony, this is what actually happened in the New Haven wreck. The rear flagman of the Bar Harbor train went back as soon as his train stopped, and had gone three or four hundred feet when his engineer whistled to him to return. He put torpedoes on the rail, as the rules require and started back, but the following train came in sight through the fog before he reached the train. Even if he had disregarded the whistle signal and continued back, he could not have gone far enough to have warned the following train in time.

It seems to be strangely overlooked by those who cling to back flagging as an additional protection under the block signal that *a time-interval rule is absolutely essential for use in connection with back flagging if it is to be of any reliance as a safeguard.*

Under the time-interval rule, a train must not leave a station within less than ten minutes of the departure of the train ahead. This gives some margin of time, if a train is stopped between stations, for the rear brakeman to get back and protect his train. But when block signals are established and the time interval between trains is abolished, the cases are frequent where a brakeman has not time to go back before the train following is upon him.

Engineers who have given most careful study to block-signal practice on American railways are agreed that equally important with the extension of block operation to the large mileage of roads still without any form of blocking trains apart is the betterment of signal practice and signal discipline on many of the railways which report that their lines are block-signaled. The signals on the New Haven-Springfield line above described are an amazing illustration of the extent to which block-signal practice on one of the most important and wealthy railways in the country, carrying a passenger traffic as dense probably as any road in the United States, is in need of radical reform.

It is indeed pretty nearly a libel on respectable block-signal practice to say that the New Haven collision occurred under the block system. For the essential principle of the block system is *the space interval between trains*. The cardinal rule is that two trains must never approach nearer each other than the length of one block. But on the section of the New Haven road where this disaster occurred the space-interval principle was ignored. A train might stop only 10 ft. beyond the block signal and a following train approach until it reached the signal. Not only that but "permissive" blocking was in force. An engineer finding a block signal at danger was obliged to bring his train to a stop but was then permitted to proceed with his train "under control," a term well known to be most elastic.

There are railways which have had the excuse of poverty for not adopting the block system. The New Haven management can plead no such excuse. It has expended enough millions in the purchase of expensive cats and dogs during the past decade to have equipped every mile of its entire system with signals conforming to the latest and best practice in signal engineering, here or abroad. In fact its outside ventures in high finance would have paid for such signal equipment ten times over.

It is fair to state that work is now in progress replacing these signals with automatic semaphores of modern type fully equipped with distant signals, a contract for the new signals having been closed several months ago. It appears, however, that the present signals have been the subject of formal protest by the engineers running on this division, through the Locomotive Engineers Broth

erhood, and also that the Connecticut Railroad Commission had urged their replacement. It does not appear, therefore, that the New Haven management adopted this improvement until it was put under pressure.

One excuse the New Haven management may indeed plead: that is that its signal practice is no worse than that of many other American railways. It is true, unfortunately, that such collisions as the fatal one at North Haven are all too common on American railways, even with all the progress made in signaling during the past 30 years. But if this in any degree lightens the responsibility of the New Haven officials, in the same degree it makes the lesson taught by those tragic deaths on the morning of Sept. 2, a lesson which will come home with force to every railway management guilty of similar neglect in safeguarding its train movements.

LETTERS TO THE EDITOR

The Proposed Improvement of an Ocean Bar at Atlantic City, N. J.

Sir—Having read with interest Major Raymond's description of the proposed mode of opening the channel at Absecon Inlet in your issue of July 24,* the undersigned feels impelled in the interest of the profession and the public to offer a few comments, not in the way of criticism but of precaution.

In the analysis of the physical conditions prevailing and of the forces operating to obstruct the inlet the general conclusions are reached: 1. That the single curved jetty "M-X"* built out on the *leeward* side of the channel "would not produce a permanent channel across the bar and would never justify its own cost."

This conclusion is correct as it has been frequently tried under similar conditions and failed, notably at Aransas Pass, Texas, where after expending over a half million dollars the experiment was abandoned with "insignificant results." The effect being merely to push the bar seaward by arresting the drift in the channel.

2. By narrowing the inlet by the jetty "A-B,"* concerning which the author adds: "This plan would lose its popularity as soon as put into execution." Reasons are stated for this belief but history would not seem to sustain them, for in 1880, the width of the inlet was only 800 ft. and yet the depths on the bar were no less than at present. It continually shifted to leeward.

3. A *submerged* jetty on the same trace is suggested, with the comment that "Although at the first blush the scheme promises success, the action of such a submerged jetty would be more or less uncertain." This conclusion is abundantly confirmed by the experience of the '70's when they were extensively tested on the Southern coasts and were gradually raised to mean tide and finally above high water, to protect the dredged channel between them.

In short, the conclusion is reached that "Any jetty outside of the inlet must affect the littoral sand movement, and any jetty inside must interfere with the flood current to a greater extent than with the ebb."

These conclusions appear to be too sweeping, for some *outside* jetties may affect the littoral sand movements beneficially and enable the currents to cut their own automatic channel across the bar without in any wise impairing the beaches in the vicinity, as at Aransas Pass before the bar jetty was connected with the coast line; while *inside* jetties may be so placed as to concentrate the ebb currents upon the gorge as to aid in the efficiency of their work, without interfering with the ingress of the flood tide. The whole question is one of proper location and form.

The jetties built in the Columbia River by the Port of Portland were found to be of great use in the formation of the channel at small cost. The reaction jetty across the bar at Aransas Pass secured depths of over 22 ft. in a navigable channel more than 150 ft. wide at that depth, without any dredging, notwithstanding a reduction of the tidal ingress by auxiliary works constructed by the Government at great cost.

Thus the lessons of actual experience have demonstrated that a single jetty properly located and reaching only across the bar, may automatically scour out, fix and maintain an excellent navigable channel at a very low cost and even with a diurnal tide of but little more than a foot. Vessels are now using this channel for export traffic.

4. It is also suggested that the bar may be deepened quite as much by dredging an equal amount of material from the interior compartments, thus, as is supposed, increasing the tidal volume; but it is manifest that this can only be increased by enlargement of the area of the tidal basin and not by deepening of the basins within the gorge, since the quantity below low tide is constant.

5. Attention is called to the removal of a shoal in Leipsic River, Del., by the removal of bridge abutments above, thus enlarging the waterway at the site of the bridge and permitting a larger volume to enter the upper reaches. This is a well known and fundamental principle, which should not be violated; yet in the treatment of one of our great tidal rivers the stream is being throttled, as by the abutments referred to, by building dams across the back channels and throttling the flood tide, causing it to scour much greater depths where they are at present sufficient for navigation, being already over 40 ft., and causing the creation of shoals both above and below the contractions, yet the work goes on at a cost of nearly $11,000,000 in the hope that after six years the channel may be dredged to 40 ft. throughout.

6. In all the earlier reports on Absecon, dredging was regarded as an adjunct to jetties, which were enormously expensive and "the inlet was given up in despair." A later report proposed dredging alone but placed the estimate of cost at a prohibitive figure. . . . . There was nothing in the proposed dredging to promise success and

*See map in that issue, p. 152.

nothing was done to put it into execution. Under these circumstances it is now proposed to construct a light-draught dredge, "of special design to try the experiment" . . . . "Even if not successful its cost will not be wasted, for it can be assigned to work at some less difficult inlet."

7. "Sand will encroach from the north, it is true, but such sand, freshly deposited, will be readily disturbed and eroded as well as dredged." To any one familiar with the relative conditions of the sand at flood and ebb tide, this statement will cause surprise, for sand once deposited is not readily dislodged by the recurrent ebb tide, but it is the encroachment of the drift from the north which drives the channel to leeward. If, however, the drift be excluded from the channel by a stable barrier and the currents be at the same time confined from spreading over the outer section of the bar, they will have sufficient energy to not only prevent deposits but actually erode the existing shoals from the deeper waters inside to those on the outer slope, automatically and permanently.

This is not an idle theory but a demonstrated fact, at a point where the tidal energy is but about one-eighth that at Absecon Inlet and it removes the problem from any uncertainty as to results or experimentation. Moreover, no dredged channel can be a permanent one under such conditions of exposure. It is admitted that at times it will close but it can soon be reopened. These are the times of storms when the channel is vital to the safety of life and property. The cost of the dredge alone would go far toward the creation of a permanent channel and it could be secured before the dredge is built if authorized immediately.

The writer is so fully convinced of the certainty of the method of a single barrier to open an adequate channel that he stands ready to guarantee to produce one, with all necessary safeguards, to the Government, either general or local, for payments based on depths secured. As his patent for this device has expired, it is public property, and it can be applied by the Government to this site without royalties at any time if so desired, with great saving in cost over any other method which has ever been proposed.

Surely the diagnosis submitted in the article referred to gives no assurance of success and involves merely a repetition of many failures.

LEWIS M. HAUPT.

Cynwyd, Penn., July 31, 1913.

On the Value of Membership in the American Society of Civil Engineers

Sir—The editorial in your issue of Aug. 21, 1913, entitled, "Making Membership in an Engineering Society a Tangible Asset," has encouraged me to write something of my own experience.

As an Associate Member of the American Society of Civil Engineers, I have taken an interest in that society's sphere of usefulness. On my first trip to New York after becoming a member, I looked forward to a visit to the Society's headquarters. I'll confess that I innocently went over there about 7 p.m., thinking that I would take dinner there. The front door was locked, but an attendant opened it for me. The lobby looked very dim. A preoccupied and somewhat bored person had me register, and answered my questions—and none of them concerned dinner. No desire was shown to make me acquainted with the building. Nobody was there, and so I left. I have never been back, for there is nothing there to attract me when I am idle.

I cannot remember ever having been more disappointed in anything. To me the Society was the Mecca of all civil engineers. Now I make occasional calculations to see if I hadn't better resign. The money I spend in dues does me little good, and does little for anyone else—except, perhaps those who get their living directly from the Society. There is no blame attached to them. The fault is the Society's.

I wear a pin. I have tried to believe that it gave me some standing with strangers, but I am afraid few know what it is. Does reference to the fact that you are a member help much when getting a position? I think not. The Society's attitude of assuming no responsibility for its members or their utterances tends to discourage the idea that membership means much.

Let the American Society of Civil Engineers wake up and take a stand for its *members,* and then *membership* will mean something. Today membership is taken more with the attitude that "it might hurt me if I didn't have it."

The Society might well introduce a register for engineers seeking employment. It could be active in bringing together these engineers and prospective employers. It could defend or condemn its members when they are publicly charged with incompetency. It might see that the civilian engineer had a larger place in governmental engineering affairs.

The American Society of Civil Engineers is a great and honorable institution; but membership in it should be more than a decorative honor. The society should fill an important place in the life of every engineer by reason of the aid it could bring to him. I sincerely hope the time is near when the *human* side of the Society will be developed.

H. B. KIRKPATRICK.

1906 Lincoln St., Evanston, Ill., Aug. 23, 1913.

A Suggested Improvement in Sieve Standardization

Sir—Could not something be done to bring about a closer concordance in fineness determinations in cement, particularly in the case of the 200-mesh sieve?

The practice of today is to have sieves approved by the Bureau of Standards, whose examination is thorough, as far as it goes, including the count of the mesh and the measure of the diameter of the wires. This is not entirely satisfactory, however, for the reason that it is possible to obtain a widely varying fineness on the same sample with two sieves, both approved by the Bureau of Standards, the discrepancy sometimes being as great as results of 85% through one sieve and 80% through another, on the same sample of cement.

It appears to the writer that the solution would be not only to check the mesh and diameter of the wire, but also to check the sieve by making a determination with it on a large standard sample of cement, most carefully prepared and of a carefully determined fineness, and ap-

proving only sieves which agree with this standard sample within at least 1%, or furnishing a plus or minus correction (based on determinations on the standard cement sample) to be used with the sieve.

There is no question that testers and engineers are much more interested in the uniformity of the results obtained with the sieves than they are with the mere details of the count of the mesh or determinations of the diameter of the wire. Those details would necessarily have to be correct to give uniformity of results, but experience has shown that it by no means follows that sieves showing, on examination, a mesh count and a wire diameter within specifications, agree in results in actual determinations.

We are constantly having trouble throughout the United States through such differences, each laboratory invariably claiming Government approval of their sieves and that, therefore, their sieves must be correct, and we have many times taken such sieves, and with the same tester, on the same cement, found extremely wide variations, as noted.

Eastern Testing Laboratories,
BERNARD ENRIGHT, Director.
Allentown, Penn., Sept. 4, 1913.

NOTES AND QUERIES

We are informed by Mr. J. W. Cunningham, who contributed the article entitled "Fire Damage to the New Harriman Bridge," "Engineering News," Aug. 21, 1913, p. 376, that the opening in the floor mentioned in the article was probably made by the firemen in fighting the fire and that the local damage to the tower was caused by the burning of a gateman's house and not by flames going through the aforementioned hole.

⁂

New Building of the Institution of Civil Engineers—The old society house of the Institution of Civil Engineers, on Great George St., Westminster, London, England, was taken some time ago by the government war office for an extension of its building. A new site for the society house was secured also on Great George St., just opposite the old site, and a new building was designed by Mr. James Miller, Architect, 1 Victoria St. The new house is now near completion, and within the past month a few of the rooms (library, reading room and council room) were turned over by the contractors for the use of the Institution. The temporary offices at 12 Dartmouth St. will probably remain in use for a short time longer.

The new building is about 250x150 ft. in ground area, and generally three stories high. The walls (and the interior finish of the rotunda and corridors) are of portland stone. The main or north front is rendered imposing by a classical colonnade and symbolic decorative panels. The west front on Princes St. is at present not complete, but will be completed after some 12 years after the removal of a corner building held under unexpired lease.

The last portion of the ground floor contains the secretarial offices, while west of the entrance hall and rotunda are reading rooms and a council room. The main stairs leading up in the rotunda are elaborately wrought and decorated in stone. The second floor contains: at the front a large library (with additional stackroom on the floor above), on the west a great hall, 100x45 ft., and at the rear the lecture theater, capable of seating about 300. In the latter the seat benches are banked, and only a small dais is provided for the lecturer; the projection lantern is on the left wall, and the screen opposite. Folding doors allow lecture theater and great hall to be thrown into one, but the doors are so constructed when closed they are undistinguishable from the other parts of the walls of the great hall. The rooms generally are floored with wood parquetry, the corridors in marble. The structure of the building comprises steel frame, heavy brick and stone walls, and reinforced-concrete floors. The nailing strips for the wood floors are 2x2's laid about 2 in. above the concrete to give space for wire conduits, mortar being subsequently packed under the strips.

The Rear Collision on the New Haven R.R., near Wallingford, Conn.

Details of the disastrous collision on the New York, New Haven & Hartford R.R., near Wallingford, Conn., on the morning of Sept. 2, are given in our editorial columns elsewhere in this issue. The accompanying view, from a photograph taken soon after the accident, is graphic proof of the tremendous impact of the on-rushing White Mountain express.

A great deal of newspaper agitation is going on over

WRECKED PULLMAN CARS, WALLINGFORD, CONN., SEPT. 2, NEW YORK, NEW HAVEN & HARTFORD R.R.

the fact that neither train was composed of steel cars; but taking into consideration the added weight and added momentum of a fast-moving train of steel cars and the extent of the destruction wrought in the present wreck, engineers at least can understand that safety from fatalities in rear collisions is not to be found in steel cars. They could at best only reduce the number of casualties. To make railway travel safe, means must be provided of preventing collisions, and it is far easier and far less costly to do this than to make the change from wooden to steel rolling stock.

⁂

Stable Manure Collection and Disposal at Columbus, Ohio, in 1912, cost the city $8150, divided as follows: teams and labor, $7626; superintendence and inspection, $319; repairs and miscellaneous, $205. During the year 3720 loads (7440 tons or 18,600 cu.yd.) were collected. Manure sales by the city brought in a total of $4107, besides which $750 worth had to be given away during the summer months because under an order of the board of health manure cannot be stored pending a market for it. Persons who have manure collected from their premises were obliged to take out a permit at a yearly charge of $3 for one horse, $5 for two horses and $1 for each additional horse. This rule went into force on Apr. 1, 1912, and from then until the close of the year the owners of 976 horses paid the city $1665. The city's gross receipts from manure sold, permits and miscellaneous, totaled $5771 for the year; gross expense (no capital charges included), $8150; net cost, $2379. The cost of collection and disposal was $2.19 gross or $0.64 net per load and $1.10 gross and $0.32 net per ton. The foregoing statements are from the annual report of E. W. Stribling, superintendent of refuse collection, Columbus, Ohio.

Test of a Hydraulic Buffer for Railroad Terminal Stations*

The office of a buffer is to prevent a locomotive or train from overrunning its stopping point. An ideal buffer should be constructed so that during the period of its travel the pressure exerted against the train will be uniform. The buffer will thus absorb the greatest amount of work possible with the smallest maximum resistance against the train, and if it fulfills this condition the reaction will be least harmful to the equipment.

In order to obtain an idea of the performance of a hydraulic buffer, and to illustrate the conditions imposed upon equipment when striking the buffer, a number of tests were run upon one (Fig. 1) built by Ransomes & Rapier, Ltd., Ipswich, England, and installed in the Grand Central Terminal, New York City. The outfit consists of a cast-steel cylinder of 22 in. internal diameter, 380 sq.in. area, and 11 ft. working length. The cylinder is grooved to permit a variable quantity of water to pass by the piston, the amount depending upon the position of the latter, and is largest when the piston is extended in position to receive a train.

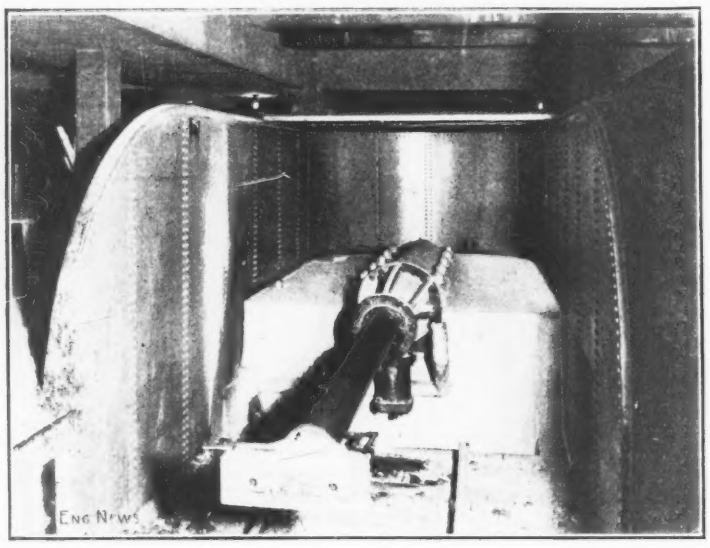


FIG. 1. GENERAL VIEW OF THE HYDRAULIC BUFFER IN POSITION

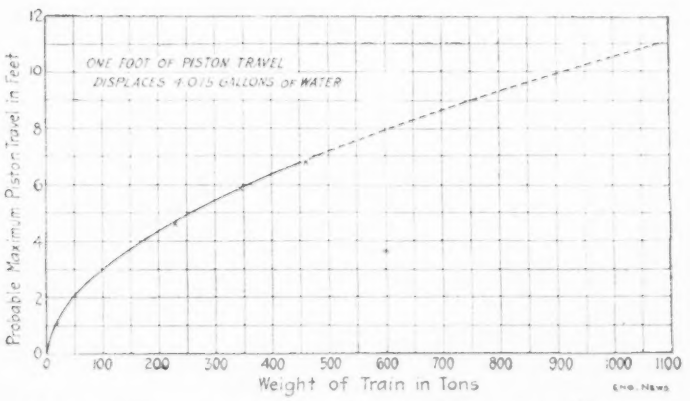

ONE FOOT OF PISTON TRAVEL
DISPLACES 4.015 GALLONS OF WATER
Probable Maximum Piston Travel in Feet
Weight of Train in Tons
ENG. NEWS

FIG. 2. WEIGHT OF TRAIN AND MAXIMUM PISTON TRAVEL OF BUFFER

The piston proper is attached to a steel ram 10 in. in diameter, extending through a stuffing box, and carrying at its extreme end a head of cast steel with a wooden protection board accurately aligned with the buffer on the locomotive. The cylinder is connected to the city water service, the pressure of which is sufficient to drive the piston out, the water discharged during the stroke being disposed of in the sewer.

The buffer is installed rigidly upon and partly imbedded in a block of concrete 20x12 ft.,x10 ft. 6 in. deep, (a total of 90 cu.yd.). It is held up on each side by five 2⅜-in. bolts extending through the foundation into the bedrock by a length varying from 6 ft. in the rear to 13 ft. in the front. The weight of the whole is approximately 195 tons.

The information required to determine the performance under different working conditions, outside of the weight of the train is principally: (a) speed of train striking; (b) pressure performance in cylinder during stroke; (c) travel of buffer piston.

The time of performance being exceedingly short, it was necessary to record the readings automatically, and a special instrument for this purpose, of the curve-drawing type, was built by an instrument manufacturer on specifications prepared by the writer.

The highest speed tested was 8.10 miles per hour, at which the maximum cylinder pressure was found to be 1135 lb. per sq.in., corresponding to a total resistance of 431,000 lb., 18,000 lb. of which was balanced by back pressure leaving 413,000 lb. effective to stop the train. All readings applying to the light locomotive fall almost exactly on the curve which has been extended to show the probable pressure at higher speeds. The readings applying to trains do not coincide as closely with the curve for the reason that the car couplings and the swinging of the cars back and forth had an erratic influence.

The curves in Fig. 4 show that above 6 mi. per hr.

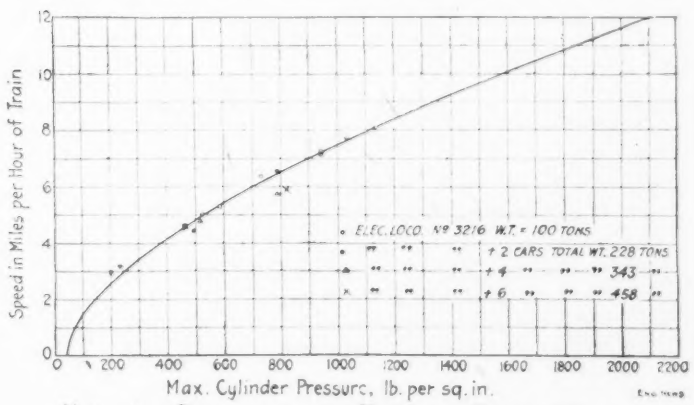

Speed in Miles per Hour of Train
Max. Cylinder Pressure, lb. per sq. in.
ELEC. LOCO. Nº 3216 W.T. = 100 TONS
+ 2 CARS TOTAL WT. 228 TONS
+ 4 " " 343 "
+ 6 " " 458 "
ENG. NEWS

FIG. 3. PRESSURE IN BUFFER CYLINDER VS. SPEED OF TRAIN

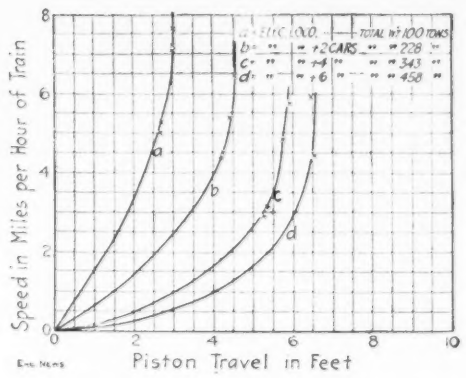


FIG. 4. WEIGHT AND SPEED OF TRAIN AND PISTON TRAVEL OF BUFFER

*Abstract of a paper read by Carl Schwartz, First Assistant Engineer, New York Central & Hudson River R.R. Co., at the May, 1913, meeting of the American Society of Mechanical Engineers, at Baltimore.

the speed of the train has practically no influence upon the travel of the piston.

These curves, in conjunction with that in Fig. 3, show that the impact and pressure against the train depend largely on its speed and that the piston travel is principally a function of the train weight. It is estimated (Fig. 2) that a train weight of 1000 tons will drive the buffer piston 10 to 11 ft., or about the total travel for which the buffer is constructed.

A 30,000-Ton Submerged Coal-Storage Pit*

The advantages of storing coal under water, wherever a large reserve supply has to be kept, have been generally recognized for several years,† though few large plants are seen yet. Probably the most notable example of submerged storage heretofore has been at the Hawthorne shops of the Western Electric Co., in Chicago, where the pits have a maximum capacity of 14,000 tons.

What is probably the largest pit so far built has recently been completed by the Indianapolis Light & Heat Co., where it is found advisable to keep a large supply of fuel on hand as protection against interference with the public service by strikes or car shortages. The total storage capacity is 30,000 tons, of which 13,000 tons are below the water level (kept at the ground level).

There is first a large concrete pit, as shown in the accompanying figure, 300 ft. long, 100 ft. wide and 28 ft. deep. It is 18 ft. below ground level, having a 10-ft. wall rising above the ground. Below the ground the wall slopes at an angle of 45°, this being done only to reduce the cost of construction. This gives a pit 72 ft. wide and 272 ft. long on the bottom. The concrete is from 12 to 18 in. thick and is reinforced throughout with twisted ½-in. square rods and is designed to withstand any pressure from water that may rise in the gravel around the pit.

Through the middle of the pit lengthways, there is a row of concrete piers placed 15 ft. apart on centers. These piers support a trestle on which there runs a standard-gage track. A 15-ton locomotive crane travels back and forth on this trestle handling the coal on both sides with a 2-yd. coal grab bucket, suspended from a 40-ft. boom. The crane and bucket are operated by one man from his stand in the cab. The crane, being self-propelled and equipped with M. C. B.-standard trucks, couplers and steam brake, switches the cars on and off the trestle. The loaded coal cars are run out on the trestle with the crane and dumped: when coal is required in the power house it is loaded into cars by the crane and carried to the boilers.

COAL-STORAGE PIT AND LOCOMOTIVE CRANE OF THE INDIANAPOLIS LIGHT & HEAT CO.

*From information furnished by the Brown Hoisting Machinery Co., Cleveland, Ohio.

†See discussions in "Engineering News," July 5 and Aug. 23, 1906, pp. 15 and 203; Dec. 24, 1908, p. 729; Aug. 18, 1910, p. 179.

Forty-pound rails are imbedded in the bottom of the pit, on 18-in. centers, with ½-in. of the head standing above the surface of the concrete. When the grab bucket is working on the bottom of the pit, the rails prevent the bucket from striking the concrete.

Water is pumped into the pit by a 4-in. centrifugal pump, taking the water from the power-house supply well, and the water enters the pit at the ground level. Through the center of the pit and beneath the trestle there is a trough 12 ft. wide and 1 ft. deep which serves as a drain. It is fitted with a drain-pipe which carries the water to either of two man-holes, one at each end of the pit. The outlet openings are protected by gratings to prevent any coal from entering the drain-pipe.

The pit was designed by Thos. A. Wynne, vice-president of the Indianapolis Light & Heat Co., and was built under his supervision. The coal handling machinery was furnished by the Brown Hoisting Machinery Co., of Cleveland, Ohio.

⁂

The Boilers Exploded on the Ohio River towboat "Alice," at Dam 2, Coraopolis, Penn., Aug. 30, killing eight of the crew. The boat sank within 10 minutes.

A Train Was Derailed and rolled down a 10-ft. embankment on the Decorah branch of the Rock Island Southern Ry., two miles from Maynard, Iowa, Sept. 1. The train was traveling at a speed of about 25 mi. per hr.; three persons were killed and many more injured.

A Large Section of a Department Store Collapsed at Peterboro, Ont., Can., Aug. 28, carrying with it many employees and customers; five were killed and seven injured. The store was occupied by the J. C. Turnbull Co. The cause of the disaster seems to have been the weakening of the structure due to extensive interior alterations involving the removal of a brick partition separating what formerly were two stores.

The Stability of the Aëroplane when under the control of a cool and experienced driver has been demonstrated to a certain extent by the remarkable feat of the French aviator Pegoud, at the aërodrome at Buc, France, when according to press dispatches he repeated, on Sept. 2, his performance of Sept. 1, of flying upside down. After circling upward to a height of 3000 ft. he dropped down 1500 ft. where, without swerving, the machine turned over, the wheels uppermost for 10 sec. The flight was witnessed by a representative of the French Army Aëronautical department.

Borings for the New Thames River Bridge—Preliminary borings for a new four-track bridge for the New York, New Haven & Hartford R.R. over the Thames River at New London, Conn., have been progressing since the early part of the summer. It has been practically decided to place the new bridge 134 ft. north of the present structure, where bed rock has been found 90 ft. below the high-water mark. It is proposed to construct the piers without the use of caisson foundations, by the use of piles resting on the bed rock. The piers will be of 4-track width, but the bridge superstructure will be first built for but two tracks. The type of draw span has not been decided upon. The estimated cost is $4,000,000.

A Rear Collision between the two sections of the Scotland-London express occurred on the Midland Ry., near Hawes Junction, Sept. 2, killing 14 persons and injuring 30 others. The two trains were south-bound for London; the first section stopped to get up steam for climbing a steep grade when the train following crashed into it, telescoping several cars, which burst into flames. When the first train came to a stop the conductor is said to have run back with a red lantern and placed torpedoes on the rails; the other section, however, failed to observe the signals in time to prevent the collision. The engineer of the colliding train stated that he was busy oiling and did not notice the signals at Mallerstand, two miles from the scene of the wreck, which the signalman at that point declared were set against him.

A Fire at Hot Springs, Ark., Sept. 5, destroyed 30 blocks of the residence district and many business and public buildings. The fire originated in a negro dwelling on Church St., near the Government Reservation, and swept southeast away from the reservation and the Army and Navy Hospital for about an hour when the strong wind that was blowing shifted in the direction of the business section, causing the destruction of the Public Utilities plant and every business structure on Malvern Ave.

Fifteen minutes after the fire started it was beyond the control of the Hot Springs firemen; special trains brought additional apparatus from Little Rock, but water, dynamite, and every means of fighting fire had failed to check the flames six hours after the fire started. Firemen many times endeavored to destroy buildings yet untouched, but the flames swept over the partially wrecked buildings and broke out anew. No fatalities were reported.

The Negative-Head or Down-Draft Water-Filter Patents have been upheld by the United States Circuit Court for the middle district of Pennsylvania in a suit brought in 1908 by the New York Continental Jewell Filtration Co. against the city of Harrisburg, Penn. Two patents were involved: No. 644,137, dated Feb. 27, 1900, and Reissue No. 11,672, dated Jan. 28, 1898 (original No. 546,738, dated Sept. 24, 1895). The Harrisburg mechanical filtration plant, on which this infringement suit was based, was built in 1905, with James H. Fuertes, consulting engineer, New York City, as engineer. Besides Mr. Fuertes, Geo. W. Fuller and Allen Hazen, both of New York City, appeared as engineers in defense of the suit. On the other side appeared Geo. A. Johnson, of New York City, as engineering expert, and Profs. Wm. P. Mason and James M. Caird, as chemical and bacteriological experts. A number of cities and water companies aided Harrisburg in the suit, including New Orleans, La., and Columbus, Ohio, and the Louisville, (Ky.) and Hackensack (N. J.) water companies. The plaintiff filter company was supported by various filter-manufacturing companies (licensees, however, of the plaintiff), including the Norwood Engineering Co., the Pittsburgh Filter Manufacturing Co. and the Roberts Filter Manufacturing Co. Robert E. Milligan, general manager of the complainant filter company, also testified in the case. The decision is of unusual interest to many cities, engineers and contractors because if sustained on appeal it will affect practically all cities and water companies in the United States, which are using so called modern rapid sand filtration plants (that is, mechanical filters built during the past nine or ten years.) Those deeply concerned with filtration will doubtless wish to see the complete decision. Copies of it may be obtained from the New York Continental Jewell Filtration Co., 15 Broad St., New York City, or presumably from the Clerk of the United States Court for the Middle District of Pennsylvania, Harrisburg, Penn.

A Heavy Rainfall at New York City, Sept. 5, caused a great congestion of traffic due to the flooding of certain sections of the Interborough subway system. The rain commenced at 10 p.m., Sept. 4, and at 12:34 the water had risen to such a height on the tracks that it was necessary to shut off the electric power. Service was discontinued between the Grand Central Station and 96th St. from 12:34 to 7:15 a.m.

The normal level of the sewers throughout the city is 13 ft. below the street surface, but when the subway was constructed it was necessary to run a sewer beneath it in order to take care of the Knickerbocker Hotel. Hence, the sewer at this point and from 42d to 45th Sts., on Broadway, is about 25 ft. below street level, siphons not being employed under the subway. The sewers also cross under the subway at 45th St. and Broadway. Certain buildings along this line (notably the Long Acre) have had their sewer connections run at this level and have not provided them with check valves. As this sewer is designed to operate, under extreme conditions, under a head, the tendency at such times is for the sewage to back up and overflow into the several basements; and in the case of the recent heavy rains this actually occurred. The worst point of entry for the water was from the basement of the Long Acre building to the platform of the Times Sq. subway station, situated at that point, and from there to the tracks. The volume of the water was increased by that which flowed down the steps of the kiosks due to the inadequacy of the catch basins in the street. The subway drainage system is provided with check-valves and was therefore of no avail in lowering the water.

This same trouble occurred during the heavy rains on July 28, 1913. The following table is a comparison of the two rainfalls over equal periods on each of the two days.

Time, minutes	Inches of Rainfall at Central Park	
	Sept. 5	July 28
5	0.60	0.51
10	1.15	0.96
15	1.59	1.20
30	2.62	1.48
60	3.31	2.73
120	3.70	3.11
24 hr.	4.14	3.11

PERSONALS

Mr. Howard S. Reed, Assoc. M. Am. Soc. C. E., has resigned his position as Engineer, U. S. Reclamation Service, at Phoenix, Ariz.

Mr. Charles E. Raynes, Mechanical Engineer, of New York City, has joined the engineering staff of Mr. Henry Floy, M. Am. Soc. C. E., Consulting Engineer, New York City.

Mr. C. E. Weaver, Roadmaster of the Illinois Central R.R., at New Orleans, La., has been promoted to be District Engineer of the Southern lines of the company with headquarters at New Orleans, La.

Mr. L. C. Brown has been promoted to be Assistant General Manager of the United Rys. of Yucatan, at Merida, Yucatan, Mexico. For the past six years he has been chief clerk to the General Manager.

Mr. J. C. Clifford, recently Roadmaster of the Illinois Central R.R., at Carbondale, Ill., has been promoted to be District Engineer of the Northern lines of the company, with headquarters at Chicago, Ill.

Mr. Charles E. Dewey, an architect, has become a member of the firm of Staege & Bennett, Engineers, Watertown, N. Y., which will hereafter be known as Staege-Bennett & Dewey, Engineers and Architects.

Mr. W. B. Thomson, Assoc. M. Am. Soc. C. E., of the engineering department of the Wheeling & Lake Erie R.R., has been appointed Valuation Engineer, with headquarters in the Electric Bldg., Cleveland, Ohio.

Mr. J. L. Harrop, recently connected with the engineering staff of the Wisconsin Railroad Commission, has been appointed Chief Engineer of the Missouri State Public Utility Commission, at a salary of $3000 per annum.

Mr. F. M. Bisbee, M. Am. Soc. C. E., Chief Engineer of the Western lines of the Atchison, Topeka & Santa Fé Ry., at Amarillo, Tex., has been appointed Chief Engineer of the St. Louis, Rocky Mountain & Pacific Ry. also.

Mr. Morris A. Zook, M. Am. Soc. C. E., Consulting Engineer, of Plainfield, N. J., has been appointed Engineer in charge of the valuation of the Grand Trunk-Wabash joint lines in Canada. He will make his headquarters in Buffalo, N. Y.

Mr. D. W. Thrower, Roadmaster of the Illinois Central R.R., at Chicago, Ill., has been promoted to be Assistant Engineer of Maintenance-of-way, with office at Chicago, succeeding Mr. A. F. Blaess, whose promotion is noted elsewhere.

Mr. A. F. Moursund, recently Assistant Superintendent of the Arizona Eastern R.R., at Globe, Ariz., has been appointed Assistant Superintendent of the Galveston, Harrisburg & San Antonio Ry., at San Antonio, Tex., succeeding Mr. T. F. Sharp, resigned.

Mr. A. F. Blaess, recently Assistant Engineer of Maintenance-of-way of the Illinois Central R.R. and the Yazoo & Mississippi Valley R.R., at Chicago, Ill., has been promoted to be District Engineer of the Yazoo & Mississippi Valley R.R., at Memphis, Tenn.

Mr. A. W. Carpenter, M. Am. Soc. C. E., Engineer of Structures, Exterior Zone, of the New York Central & Hudson River R.R., at New York City, has been relieved of his duties in this office in order to devote his whole time as Engineer in charge of the valuation of the railway's property.

Mr. A. C. Mann, recently Purchasing Agent of the Central of Georgia Ry., has been appointed Purchasing Agent of the Illinois Central R.R., at Chicago, Ill., succeeding Mr. J. C. Kuhns, resigned. Mr. Mann is succeeded on the Central of Georgia Ry., by Mr. J. L. Bennett, formerly General Storekeeper.

Mr. George L. Bean, M. Am. Soc. C. E., who recently resigned his position with the American Water Works & Guaranty Co., has accepted a position with Mr. Henry Floy, M. Am. Soc. C. E., Consulting Engineer, New York City, to assist in the valuation of public-utility properties, particularly water-works plants. Mr. Bean was with the National Board of Fire Underwriters as Hydraulic Engineer for several years previous to his connection with the American Water Works & Guaranty Co.

Mr. George W. Abbott has been appointed Division Engineer of the Boston division of the Boston & Albany R.R., with office at Boston, Mass., succeeding the late E. A. Haskell, whose death is noted in our obituary columns of this week. Mr. Abbott has been with the engineering department of the Boston & Albany R.R. since 1907, and has been promoted through the grades of Assistant Engineer, Assistant Designing Engineer and Acting Division Engineer. He was born in Syracuse, N. Y., in 1876 and entered the railway service with the New York Central Lines in 1901.

Mr. H. T. Welty, recently Engineer of Bridges, West Side Improvements, New York City, of the New York Central & Hudson River R.R., has been appointed Engineer of Structures, Exterior Zone, with headquarters at the Grand Central Terminal, New York, succeeding Mr. A. W. Carpenter, M. Am. Soc. C. E., promoted as noted elsewhere. Mr. Welty is a graduate of the Case School of Applied Science, class of 1902, and his service with the New York Central & Hudson River R.R. began as a bridge draftsman. Since then he has been Assistant Engineer in the bridge department, Assistant Engineer of Structures, Engineer of Bridges, maintenance-of-way department, and Bridge Engineer, West Side Improvements.

OBITUARY

Samuel Tate, Engineer and promoter of railways in Central and South America, committed suicide, Sept. 3, in a hotel at Asheville, N. C. He is survived by a widow and two daughters.

Jennings S. Cox, Jr., M. Am. Inst. M. E., died in New York City, Aug. 31. He was a graduate of the School of Mines, Columbia University, class of 1887, and for the past 18 years had been General Manager of the Spanish-American Iron Co., at Santiago de Cuba.

Fred Harris Daniels, M. Am. Soc. M. E., Chief Engineer of the American Steel & Wire Co., died at his home in Worcester, Mass., Aug. 31. He was 60 years old and famous the world over for his inventions of wire-making machinery. A biographical sketch will appear in a later issue.

Major Hubert L. Wigmore, Corps of Engineers, U. S. A., Military Attaché to the American Embassy at Tokio, Japan, died Sept. 2, aged 39 years. Major Wigmore graduated from the United States Military Academy at West Point in 1899, and after serving three years in Cavalry Service was appointed First-Lieutenant, Corps of Engineers, in 1902. Most of his engineer service was in the Philippine Islands and at Washington, D. C.

Edward Aaron Haskell, Division Engineer of the Boston division of the Boston & Albany R.R., died Aug. 24, at his home in Newtonville, Mass. He was born in Maine in 1864 and attended the Massachusetts Institute of Technology from 1883 to 1885. He entered the service of the engineering department of the Boston & Albany R.R. about 1890. In 1894 he was promoted to be Division Roadmaster at Springfield, and later at Pittsfield, Mass. In 1907 he was made Division Engineer at Boston.

C. M. Taylor, former Mechanical Superintendent of the Second district of the Chicago, Rock Island & Pacific Ry., at Topeka, Kan., died Sept. 3, at Colorado Springs, Colo. He was granted leave of absence in July on account of illness. He was born in 1852 and entered the service of the Rock Island Lines in 1906 as District Master Mechanic and Mechanical Superintendent. He had previously been Mechanical Superintendent of the Western grand division of the Atchison, Topeka & Santa Fé Ry., at La Junta, Colo.

John S. Cook, Master Mechanic of the Georgia R.R., at Augusta, Ga., died Aug. 28, at the Johns Hopkins Hospital, Baltimore, Md., aged 86 years. He was born in Brooklyn, N. Y., in 1827 and at 22 years of age became a machinist in the shops of the Central R.R., at Savannah, Ga., having previously served an apprenticeship at the Baldwin Locomotive Works in Philadelphia, Penn. Later he was a machinist, locomotive engineer, foreman and Master Mechanic of the Georgia R.R., serving continuously from 1850 to the time of his recent illness.

ENGINEERING SOCIETIES

COMING MEETINGS

ROADMASTERS' AND MAINTENANCE OF WAY ASSOCIATION.
Sept. 8-12. Annual convention at Chicago, Ill. Secy., L. C. Ryan, Chicago and North Western Ry., Stirling, Ill.

AMERICAN CHEMICAL SOCIETY.
Sept. 8-12. Annual meeting at Rochester, N. Y. Secy., Chas. L. Parsons, Durham, N. H.

AMERICAN ELECTROCHEMICAL SOCIETY.
Sept. 9-11. The 24th general meeting at Denver. Secy., Prof. Jos. W. Richards, Lehigh University, South Bethlehem, Penn.

NEW ENGLAND WATER-WORKS ASSOCIATION.
Sept. 10-12. Annual convention at Philadelphia, Penn. Secy., Willard Kent, Narragansett Pier, R. I.

AMERICAN PUBLIC HEALTH ASSOCIATION.
Sept. 9-13. Annual meeting at Colorado Springs, Colo. Secy., S. M. Gunn, 755 Boylston St., Boston.

MASTER CAR AND LOCOMOTIVE PAINTERS' ASSOCIATION.
Sept. 9-12. Annual convention at Ottawa, Ont. Secy., A. P. Dane, Reading, Mass.

NATIONAL PAVING BRICK MANUFACTURERS ASSOCIATION.
Sept. 17-18. Tenth annual meeting at Cleveland, Ohio. Secy., W. P. Blair, 824 Engineers' Building, Cleveland, Ohio.

INTERNATIONAL CONGRESS OF REFRIGERATION.
Sept. 17-Oct. 1. Third congress, at Chicago, Ill. Secy., J. F. Nickerson, Chicago.

ILLUMINATING ENGINEERING SOCIETY.
Sept. 22-26. Annual convention at Pittsburgh, Penn. Secy., I. D. Israel, 29 W. 39th St., New York City.

AMERICAN MINE SAFETY ASSOCIATION.
Sept. 22-24. Annual meeting at Pittsburgh, Penn. Chairman, H. M. Wilson, 40th and Butler Sts., Pittsburgh, Penn.

AMERICAN ROAD CONGRESS.
Sept. 29-Oct. 4. Annual convention at Detroit, Mich. Gen. Secy., J. E. Pennybacker, Colorado Bldg., Washington, D. C.

AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.
Oct. 7-10. Annual convention at Wilmington, Del. Secy., Geo. H. McGovern, Chamber of Commerce, Wilmington, Del.

RAILWAY SIGNAL ASSOCIATION.
Oct. 14-16. Annual meeting at Nashville, Tenn. Secy., C. C. Rosenberg, Times Building, Bethlehem, Penn.

AMERICAN ELECTRIC RAILWAY ASSOCIATION.
Oct. 13-17. Annual convention at Atlantic City, N. J. Secy., H. C. Donecker, 29 West 39th St., New York City.

AMERICAN GAS INSTITUTE.
Oct. 15-17. Annual meeting at Richmond, Va. Secy., Geo. G. Ramsdell, 29 West 39th St., New York City.

AMERICAN MINING CONGRESS.
Oct. 20-25. Annual convention at Philadelphia, Penn. Secy., J. Callbreath, Majestic Building, Denver, Colo.

AMERICAN RAILWAY BRIDGE AND BUILDING ASSOCIATION.
Oct. 21-23. Annual convention at Montreal, Que. Secy., C. A. Lichty, 319 North Waller Ave., Chicago, Ill.

NATIONAL ASSOCIATION OF RAILWAY COMMISSIONERS.
Oct. 28. Annual Convention at Washington, D. C. Secy., Wm. H. Connolly, Interstate Commerce Commission, Washington, D. C.

International Association of Fire Engineers—The 41st convention, held in the Grand Central Palace, New York City, Sept. 1-6, was notable as having a larger attendance and exhibition of fire-fighting apparatus than any previous convention of the association. Papers were presented discussing: "Durability, Efficiency and Economy of Operation of Motor Apparatus," by Chief A. V. Bennet, Knoxville, Tenn.; "Fire Insurance and Its Relation to Incendiarism," by Fire Commissioner Joseph Johnson, New York City; a paper (from the insurance-company standpoint) in opposition to that of Mr. Johnson, by Frank Lock, New York; "Fire Prevention and Uniform State Marshall Laws," T. J. Ahearn, Fire Marshall of New York State; "The Gasoline Motor Pumping Engine," Chas. S. Demarest, Chief of Construction, New York Fire Dept.

Other features of the program included a fireboat demonstration, a fireman's parade and the unveiling of "Firemen's Memorial," at Riverside Drive and 100th St.

American Road Congress—The third annual congress will be held at Detroit, Mich., Sept. 29-Oct. 4. Papers will be presented on the subjects of brick, concrete, gravel, dirt and unsurfaced roads. Other papers are: "Financing Road Improvements"; "Drainage Structures" and "Selection of Road Material." An exhibit of road-building materials and machinery will be held in the Wayne Gardens.

Pacific Association of Consulting Engineers—The annual meeting will be held at the University Club, San Francisco, Calif., Sept. 19. President Marx, who was the delegate of the club to the Congress of Consulting Engineers, at Ghent, Belgium, July 18-21, will deliver an address.

American Association of Railway Superintendents—The annual meeting was held in Chicago on Aug. 21-22. Committee reports were presented on proposed changes in the standard code of train rules and in the rules governing interchange car inspection. A paper on "Immigration," by G. B. Schmitt (Chicago, Rock Island & Pacific Ry.) opposed restrictions now proposed; and one on "Railway Economies" was read by W. W. Wheatley. The former dealt particularly with the subject of effective use of freight cars and the necessity of greatly improving the present conditions of short daily mileage and large detentions at yards and terminals.

Officers were elected as follows: President, Charles Burlingame (Wiggins Ferry Co.), St. Louis, Mo.; vice-president, H. R. Sanders (Chicago, Rock Island & Pacific Ry.), Kansas City, Mo.; secretary, E. H. Harman (Terminal R.R. Association), St. Louis, Mo. The time for the annual meeting was changed by vote of the association, and the next meeting will be held in May, 1914.

Railway Signal Association—The 17th annual meeting will be held at Nashville, Tenn., on Oct. 14-16 and at Chattanooga, Oct. 17. At Chattanooga arrangements have been made for special trips to Lookout Mountain, Chickamauga Park and the battlefield on Missionary Ridge. Special trains to Nashville will be run from Chicago and New York.

Boiler Manufacturers Association—The 25th annual convention was held in Cleveland, Ohio, Sept 1-4, 1913. An elaborate program of entertainment for guests was laid out by the local members and the convention closed with a banquet.

On the first day, the visitors were welcomed to the city by the Mayor, Newton D. Baker. After the exchange of felicities the meeting was addressed by A. V. Cannon, member of the National Conference on Uniform Laws and the Committee on Uniform Boiler Specifications of that conference. His argument was that it was advisable to have a uniform boiler-construction law rather than a uniform boiler-inspection law. He described the ways in which this conference on uniform state laws approached such a subject and outlined the difficulties in securing a uniform law. He gave a summary of the boiler laws of the country to show their great diversity and the impossibility of securing the desired relief the manufacturers were seeking without providing specifications for standard types of boilers which might be embodied in any standard law for each state.

At the second session the subject of uniform specifications was continued. A Committee on Uniform Specifications presented a report, and its reading and discussion were listened to by the members of the National Association of Tubular Boiler Manufacturers also, which association was holding sessions in Cleveland at the same time. The report described the various activities of similar committees since 1889. It was shown, for instance, how their early attempts were to have uniform specifications embodied in the laws of various states, but these attempts were abandoned for an educational campaign in boiler construction and in an adequate inspection law. The committee recommended the adoption of the 1912 report with a couple of slight corrections. It was also recommended that a model short law should be drafted and that a special committee of five should handle the matter in connection with the Committee on Uniform Specifications. The report was discussed by members of both associations and by representatives of the inspection boards.

A paper by T. E. Durban, of Erie, Penn., discribed some of the problems which had been encountered in seeking uniform specifications and outlined the bearing of existing diverse laws on a large number of practical points in boiler construction.

At this session there was read a paper by W. C. Connelly, of Cleveland, on "Shop Costs." In this he particularly called attention to the failure of many manufacturers to include the proper amounts of overhead charges in their quotations on products, the items often forgotten being 6% interest on capital invested in plants and 4% depreciation on buildings with 15% on equipment. He advocated the distribution of overhead burden in proportion to the labor charge.

The paper "Factory Efficiency," by L. I. Yeomans, of Chicago, was a discussion of the several factors which enter into the management of manufacturing plants so as to secure best possible results. The author described himself as being conservative in his ideas of factory management and even made the statement that he had been unable to find a single new element in the so called scientific management doctrine. His discussion followed somewhat familiar lines except that he did not approve so called profit sharing, which it seemed to him had all the appearance of a division of spoils rather than compensation for effort. The discussion turned out to be a lively tilt over charges that manufacturers too readily accepted a "standard scale" of wages irrespective of individual merit, passing the burden on to the customers.

The next convention will be held in New York City some time in September, 1914. The president, secretary and treasurer remain the same as last year. The list of officers elected follows: President, E. D. Meier, New York, N. Y.; Secretary, J. D. Farasey, Cleveland, Ohio; Treasurer, Jos. F. Wangler, St. Louis, Mo.; Vice-Presidents, T. M. Rees, Pittsburgh, Penn.; J. Don Smith, Charleston, S. C.; H. D. MacKinnon, Bay City, Mich.; L. E. Connelly, Cleveland, Ohio; R. Joy, Oswego, N. Y.